BEHAVIOUR AND EFFECTIVENESS OF IN-GROUND SUSTAINABLE URBAN DRAINAGE SYSTEMS IN SCOTLAND

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A thesis submitted in partial fulfilment of the requirements of the University of Abertay Dundee for the degree of Doctor of Philosophy

This research programme was carried out in collaboration with HRWallingford and Yorkshire Water Authority

July 2005

I certify that this thesis is the true and accurate version of the thesis approved by the examiners.

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Date 28th July 2005

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## ABSTRACT

Infiltration trenches and filter drains are the most common types of sustainable urban drainage systems (SUDS) in Scotland. Despite their extensive use there has been only limited examination of their performance, with the general expectation that failure through lack of maintenance and poor detailing design would necessitate reconstruction within a limited time period.

This research worked towards enhanced detailing and improved operation and maintenance of in-ground SUDS. It focused on information gained from on-site monitoring of three filter drain and three infiltration trench systems and combined the outcomes with information gathered from some 40 assessments of in-situ systems in Eastern Scotland. Performance results were produced using a newly developed scoring system, named the Schlüter Score, and results showed good performance at only 19% of systems; 19% were rated as poor and a high failure rate of 23% was discovered. Similar results were produce from a conventional environmental risk assessment identifying more than 30% of systems which require immediate mitigation measures to reduce their environmental risk. These findings give an indication of the varied performance of systems in Scotland and also show the need for knowledge enhancement in the field of in-ground SUDS. A main outcome from this research is a list of recommendations which are applicable to design engineers, developers, and authorities and contribute to achieving optimum long-term performance in terms of outflow quality and flow attenuation.

A total number of 43 sites were investigated, the majority being systems located in small-to-medium size housing developments. The average age of the sites was 4 years, the oldest and youngest being 10 and 1 years, respectively. Almost 75% of all systems discharge to natural watercourses, disconnecting a significant amount of impermeable area from combined sewer systems. Catchment areas varied from $392m^2$ to $200{,}000m^2$, typically consisting of road and roof surfaces. High-level by-passes are used to ensure hydraulic performance and these were found at more than 50% of all systems. Maintenance programmes were generally not in place but this study showed that regular maintenance is vital for the longevity of in-ground SUDS. A significant number of systems require major upgrading before they may be considered satisfactory and a maintenance appraisal is provided for each system.

Hydraulic results from on-site monitoring showed good-to-satisfactory performance with flow volume reduction of 34-80% and peak flow reduction of 47-86%. The system's design, treatment volume and soil permeability were found to be the main influence on the hydraulic performance. Simulation of flows significantly contributed to the conclusions drawn and hydraulic simulation was carried out for each of the monitored systems. It was found that existing models did not represent in-ground filter systems adequately and an improved model was developed based on the finite-volume-method and Darcy's law. This model, which uses the acronym FVD, was developed in collaboration with HR Wallingford Ltd and enables flow-simulation through gravel-filled SUDS. The FVD model was validated using on-site monitored data and an excellent agreement with observed data was achieved. Wallingford Software have proposed to include the FVD model in their next release of Infoworks Version 6.5.

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## List of Abbreviations

| | |
|---|---|
| API | Antecedent Precipitation Index |
| ATV | Allgemeine Technische Vorschriften (General Technical Guidelines) |
| BMP | Best Management Practise |
| BOD | Biochemical oxygen demand |
| BRE | British Research Establishment |
| BS | British Standard |
| CCTV | Closed-circuit television |
| CIRIA | Construction Industry Research and Information Association |
| CP | Car Park |
| CSO | Combined Sewer Overflow |
| DEX | Dunfermline East Expansion Area |
| EA | Environment Agency |
| EPIT | End-of-Pipe Infiltration Trench |
| F | Field |
| FVD | Finite Volume D'Arcy |
| H | House Area |
| LF | Lateral Flow |
| LR | Local Road |
| MDE | Maryland Department of the Environment |
| MR | Major Road |
| OK | Offlet Kerb |
| FRPB | Forth River Purification Board |
| RSFD | Road Side Filter Drain |
| S | Soakway |
| SEPA | Scottish Environment Protection Agency |
| SSO | Storm Sewer Overflow |
| SUDS | Sustainable Urban Drainage Systems |
| SWMM | Storm Water Management Model |
| TGP | Typical Trapped Gully Pot |
| TS | Time Series |
| TSM | Rainfall Time Series Manager |
| UAD | University of Abertay Dundee |
| USSEPA | United States Environment Protection Agency |
| $V_T$ | Treatment Volume |

## List of Symbols

| | |
|---|---|
| $A_i$ | Data value (rainfall intensity or flow) multiplied by the time-step |
| $\widetilde{Area}_{(i+1)}$ | Average cross section area of cell (i+1) (m2) |
| $\Delta h_{(i)} \Delta x_{(i)}^{-1}$ | Water surface gradient |
| $\Delta H$ | Head difference |
| $\Delta x$ | Cell length |
| $\Delta t$ | Time step |
| $\mu$ | Specific yield |
| A | Cross-sectional area ($m^2$) |
| $Area_{Sides}$ | Trench cross sectional area |
| ASCII | American Code for Information Interchange |
| Cd | Cadmium |
| $C_p$ | Decay coefficient |
| Cu | Copper |
| D | Diameter (m) of the manhole |
| F | Goodness of fit criterion (%) |
| $H_{Manhole}$ | Manhole level |
| $H_{Trench}$ | Water level within the trench |
| I | Slope of water flow |
| K | Hydraulic conductivity of the soil ($ms^{-1}$) |
| $K_h$ | Hydraulic conductivity of fill material ($ms^{-1}$) |
| $K_{Sides}$ | Hydraulic conductivity of trench side-wall ($ms^{-1}$) |
| L | Length (m) |
| $M_{10-30}$ | 10year return period, 30min duration |
| n | Number of variable (events, cells, storage cascades, etc) |
| k | Retention time of storage cascade (min) |
| NTU | Nephelometric Turbidity Units |
| P | Porosity of fill material (%) |
| Pb | Lead |
| $P_{-n}$ | Depth of Rainfall (mm) on day n before the event |
| $P_{t'-9}$ | Rainfall depth between time t' and 09:00 am |
| Q | Volume of discharge rate ($m^3s^{-1}$) |
| $Q_{cell(i)}$ | Discharge from $cell_{(i)}$ to $cell_{(i+1)}$ ($m^3s^{-1}$) |
| $Q_{LossesBase}$ | Exfiltration losses from the trench bottom ($m^3s^{-1}$) |
| $Q_{LossesSides}$ | Exfiltration losses from the trench side ($m^3s^{-1}$) |
| $r_i$ | Difference between recorded and simulated volume per event (%) |
| T | Time |
| $T_{CG}$ | Time of centre gravity |
| $TD_i$ | Duration from the event start up to each data point |
| $T_{ES}$ | Event Start Time |
| $V_{Exfiltration}$ | Losses due to exfiltration ($m^3$) |
| $V_{Manhole}$ | Manhole volume |
| VT | Treatment volume |
| $V_{Trench}$ | Flow volume ($m^3$) |
| W | Width (m) |
| Zn | Zinc |

# CHAPTER 1 INTRODUCTION

In-ground SUDS are often the developer's preferred choice in urban areas in Scotland as they require little space, are inexpensive, and can permit development where sewerage capacity is limited. A rapid growth in the installation of SUDS since 1996 with a total of 4000 SUD systems at 767 sites around Scotland was reported by Wild et al. (2002). That study also showed that infiltration techniques were among the most common, with almost 500 filter drains and infiltration trenches to date, and this is expected to rise to over 1200 in-ground SUDS by 2008. Despite the extensive use which has been made of infiltration pits and trenches, or soakaways, there has been only limited examination of their performance (Warnaars et al., 1999 and Abbott & Comino-Mateos, 2001) and a general expectation that failure through blockage and inadequate maintenance would necessitate reconstruction within a limited time period (Pratt, 2001).

The hydraulic behaviour of typical in-ground SUDS was investigated for this research programme. The investigation was carried out at various levels to obtain a thorough understanding of typical systems, and also to reflect the general performance of the large number of systems installed to date. On-site monitoring of a total of six filter drain and infiltration trench systems was undertaken, in addition to a visual inspection survey of some 40 sites. This approach enabled a comparison to be made between the different systems and an identification of causes of failure.

Information about future maintenance for continuous tasks and possible reconstruction is urgently needed. This study provides information on the short as well as long-term maintenance requirements.

On-site monitoring in combination with hydraulic modelling of the six systems provided detailed understanding of each system and enabled a performance assessment of the 36 additional sites. The Stormwater Package Erwin was used to model each system under investigation. In addition to the generic Erwin models, a new model was developed using a finite volume approximation in combination with Darcy's law and one site was modelled. This model, named Finite Volume Darcy (FVD) model, was made available to HRWallingford with the intention that it will eventually be incorporated into the Infoworks software package. Figure 1-1 shows an overview of the thesis, including papers and publications, and the stages leading to the main outcomes from the thesis.

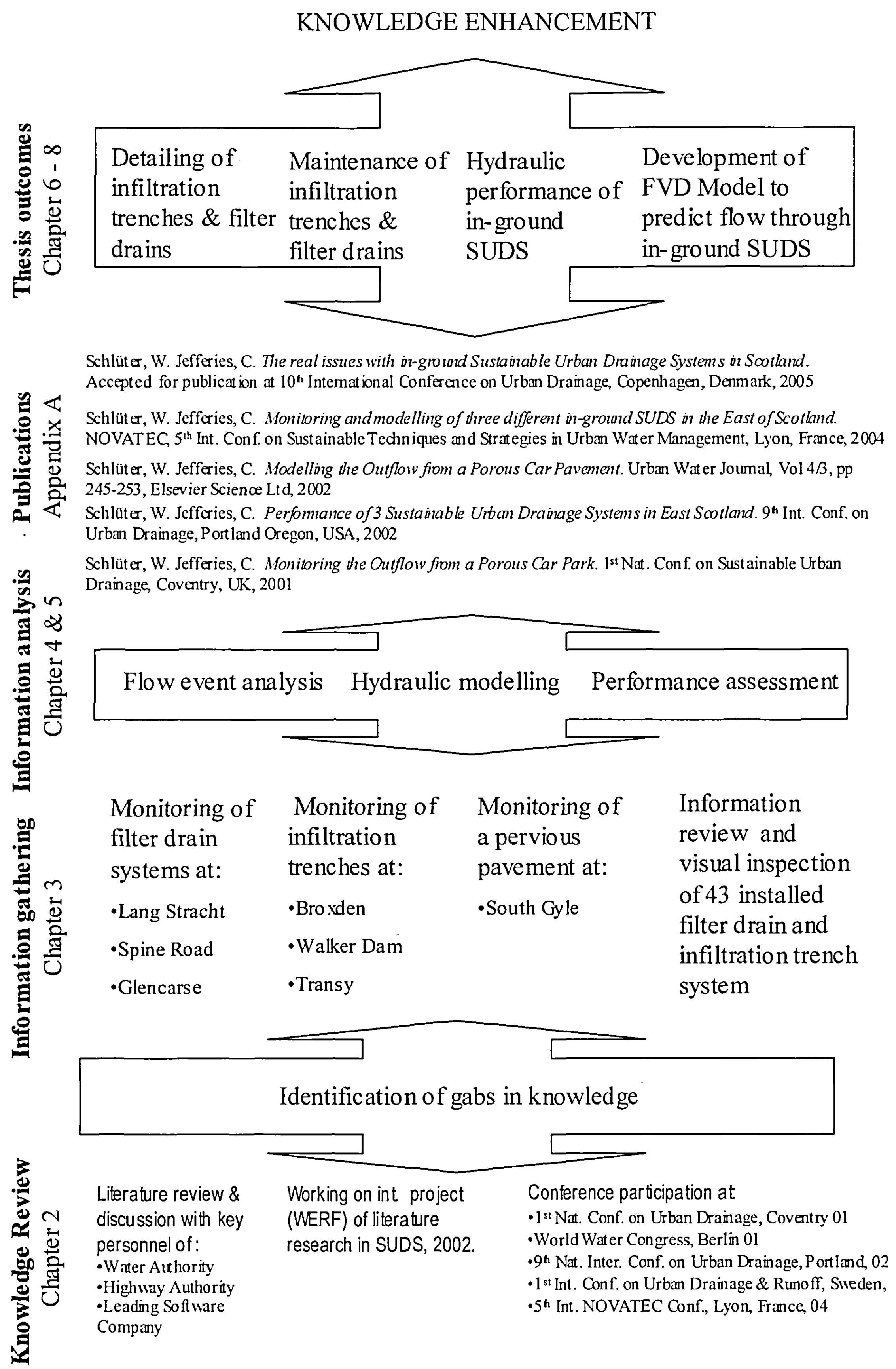



**Figure 1-1: Overview of thesis**

This research programme followed on from an MSc study on monitoring and modelling the outflow of a pervious pavement. Although results from the previous study are not repeated here, it did result in a number of publications and conference papers and it contributed to the outcomes of this thesis (Figure 1-1).

## *1.2 Rationale*

This research started from the view that a large number of the filter drain/ infiltration trench systems had been installed even though little research had been undertaken into their behaviour. It focused on information gathered from on-site monitoring of one long-term (3 years) and five short-term (3-6 months) studies, and combined the outcomes from several evaluations of filter drains and infiltration trenches in Eastern Scotland. Particular attention was given to the causes of failure of in-ground SUDS and the importance of construction management and operational maintenance. Computer simulation of flows significantly contributed to the understanding and conclusions drawn.

A wide range of in-ground systems, particularly filter drains and infiltration trenches, were studied to gain an overall understanding of their problems. This was complemented by a detailed examination of six sites to evaluate the importance of the problems raised. The Schlüter scoring system was introduced which enabled an inter-system comparison to be made with emphasis on their detailing. The systems' maintenance procedures were assessed and future maintenance requirements and possible reconstruction to restore a satisfactory operation were proposed.

Each of the systems monitored was modelled using a propriety package and an improved modelling concept was developed which is based on the system's physical characteristics, which enables flow-simulation through gravel-filled SUDS. Data from one of the detailed monitoring sites was used to validate this model.
Figure 1-2 presents an overview of the various tasks undertaken and the main areas of knowledge enhancements.

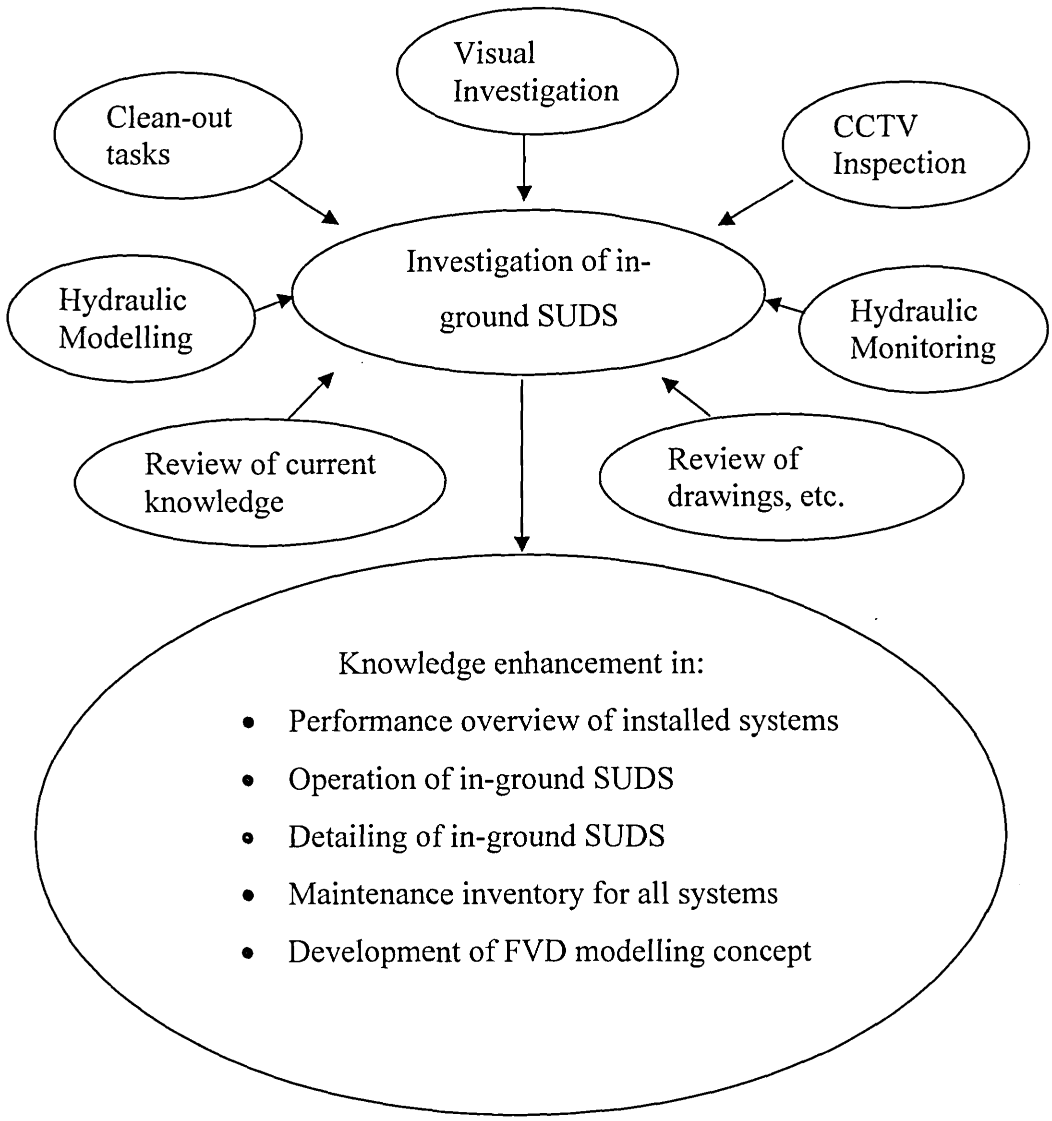



**Figure 1-2: Tasks undertaken and main outcomes**

## 1.3 Objectives & fundamental issues

This research worked towards investigating the effectiveness of in-ground sustainable urban drainage systems in Scotland by monitoring and inspecting as-built systems, and carrying out computer simulation. This comprised the following:

- Gathering and evaluating flow data from a number of typical infiltration trench and filter drain systems.
- Developing generic hydraulic models to understand the systems better.
- Collating information from a larger number of additional sites and carrying out visual inspections of installations.

Computer simulation was combined with in-depth monitoring and investigation of numerous in-situ systems. As a result, the project enhances knowledge in the field of in-ground SUDS in urbanised areas.

**The project's overall aim** was to evaluate the effectiveness of in-ground SUDS in the urban environment. The overall aim was achieved by the following objectives:

**Objective (a):** Evaluate the causes of failure of in-situ filter drain and infiltration trench installations from on-site monitoring results, visual inspections and design information.

**Objective (b):** Evaluate the systems' hydraulic performance using on-site monitoring.

**Objective (c):** Build generic models to undertake a performance comparison and develop an improved model for a more realistic representation of flow the attenuation through in-ground SUDS.

**Objective (d):** Synthesise the knowledge and experience from monitoring, modelling and onsite assessments into recommendation for improved detailing.

The philosophy of the project was to synthesise improved detailing information of in-ground SUDS and develop operational guidance which would contribute to achieving optimum long-term performance. It was hypothesised that improved detailing design and operational guidance maximises system life span and enhances their performance in terms of outflow quality and flow attenuation.

## 1.4 Principal outcomes

Experience from on-site monitoring was integrated with the visual inspection-surveys to evaluate the causes of failure from existing in-ground SUDS.

The Schlüter scoring system was introduced to enable a numerical comparison between systems and to aid identifying systems demonstrating good practice. Results show that 36% of systems had fair performance, and good performance was likely to be achieved for only 16% of systems. Only one system was considered to be performing excellently. Unfortunately, 20% were rated as poor and a high failure rate of 24% was found.

To identify possible causes of failure, several attempts were made to develop a relationship between the overall score and other parameters such as, treatment volume per drained area, number of houses connected, type of system and age. Unfortunately, there was no apparent relationship.

Systems which followed good practice featured a number of characteristics which are outlined as follows:

- Sufficient sump capacity to promote sedimentation prior to inflow into the trench system.
- Disconnection of inflow and outflow to promote filtration.
- Inspection chambers at either end of the system.
- Access to allow flushing out of debris.
- Installation of a dip plate to hold back any floating particles and chemicals and reduce the flow velocity.
- Maximise the length of the perforated inlet pipe to minimise the risk of blockage.
- Maximise elevation of any overflow to maximise storage volume, promote filtration and reduce the number of overflow events.

The findings showed that a significant number of in-ground filter drains, infiltration trenches and soakaways required major upgrading before they might be considered to be satisfactory. One-off maintenance tasks were estimated for 32 sites. The tasks range from a complete reconstruction through to substantial replacement and to cleaning out tasks. In addition to the one-off tasks, ongoing routine maintenance is required for all systems, and maintenance intervals are proposed depending on the characteristics of each system. Maintenance intervals vary from annually for major road drainage systems to once in ten years for small systems in housing estates or car parks.

Review of current knowledge gave evidence that, to date, no standardised programms were available to represent flow through in-ground SUDS. The newly developed FVD model provides a good prediction of flow through gravel-filled infiltration trench systems. The model uses the Darcy equation in combination with a finite volume approximation. It is capable of predicting flow through infiltration trenches using only their physical characteristics; i.e. the change of dimensions or material related variables result in a change of hydraulic performance. No head-discharge relationship or black box estimation is required.

On-site monitoring of typical infiltration in-ground SUDS showed good-to-satisfactory performance with average flow volume reduction of 30 to 80% per event. Average lag times were found to be between 27 and 180 minutes and one system showed satisfactory performance despite long emptying times of up to two weeks.

The treatment volume was found to be most important for extreme rainfall events. The storage volume of two infiltration trench systems was found to be under-designed by 75 to 80%. However, on-site monitoring and modelling results showed satisfactory hydraulic performance.

Computer simulation showed that an elevated outlet could significantly improve system performance for small to medium events. Approximately 40% more flow volume reduction was achieved when increasing the elevation of the outlet at a site with suitable drainage soil conditions. No significant improvement was found for extreme events or design rainfall and this may be due to the small storage volume of this system.

## 1.5 Structure of thesis

This thesis contains 8 chapters, including this **Introduction** chapter which provides the objectives, fundamental issues and principal outcomes. The chapters are:

**Chapter 2 – Review of current knowledge**

In-depth knowledge on Sustainable Urban Drainage Systems (SUDS) and the effects of urbanisation was gained through a continuous literature review and attendance at one national and four international conferences in addition to numerous meetings of specialist groups. The findings from this review of knowledge are presented in chapter two.

**Chapter 3 –Monitoring of typical systems**

This chapter presents results from the sites selected for on-site monitoring. The monitoring equipment is described at the start of the chapter and the parameters which were analysed for are described. Some background information on the study sites is given, including a detailed site description and some basic monitoring results. More detailed results are presented in Appendix C. Construction drawings of each monitoring site are included in Appendix D. Monitoring results are compared at the end of Chapter 3.

**Chapter 4 – Overall assessment of in-situ systems**

This chapter assesses the general performance of installed in-ground SUDS using the newly developed Schlüter scoring procedure. An environmental risk assessment is undertaken to provide a comparison to the Schlüter score. Results from a visual inspection are presented and problems are identified. Particular attention is given to detailing, and a separate section outlines maintenance procedures required to provide a satisfactory performance.

**Chapter 5 – Performance evaluation using a standard modelling package**

A hydraulic comparison between the monitoring sites was undertaken using the modelling package Erwin. Individual models were built for each site and the model development of one site is included at the start of chapter 5 as example. This is followed by a brief description of the modelling elements. A comparison of runoff modelling elements available in Erwin was undertaken prior to sensitivity analyses. Calibration results are then presented and a performance comparison is outlined at the end of the chapter. Hydrographs of the simulated and observed outflows at each monitoring site is presented in Appendix G.

**Chapter 6 – Development of the Finite-Volume Darcy's Flow (FVD) Model**

This chapter introduces the newly developed FVD model. Initial development used a simple Excel model of an arbitrary system and this was translated to a code-based model. Model validation was undertaken using monitored data from one of the study sites.

**Chapter 7 – Discussion**

This chapter draws together the different sections which are outlined in the thesis by discussing the inter-relations between the various findings and results.

**Chapter 8 – Findings and Conclusions**

This chapter summarises and concludes on the main findings and provides a list of recommendations. Future work is outlined at the end of chapter 8.

# CHAPTER 2 IN-GROUND SUDS IN CONTEXT OF TODAY'S STORMWATER SYSTEMS

*This chapter provides basic information about the effects of urbanisation on the water balance prior to an overview of SUDS in Scotland. A more detailed literature review on filter drains and infiltration trenches is presented thereafter. Some basic information is also given on computer simulation software and the importance of modelling to urban drainage and a brief introduction to Risk Assessment is provided. Gaps in current knowledge and how this study contributes to the enhancement of knowledge in the field of in-ground SUDS are summarised at the end of this chapter.*

## 2.1 Effects of urbanisation on the water balance

As urbanisation has intensified in the last decade and extended into previously rural areas, sealed surface areas have increased with significant impact on the water balance.
Most surfaces of natural landscapes trap rainwater and snowmelt and allow them to slowly filter into the ground. Runoff tends to reach receiving water bodies gradually. In contrast, non-porous urban landscapes, such as like roads, bridges, car parks, and buildings, do not let runoff percolate into the ground. Water remains above the surface, accumulates, and runs off in greater amounts and at a faster rate. Figure 2-1 shows these changes in run-off.

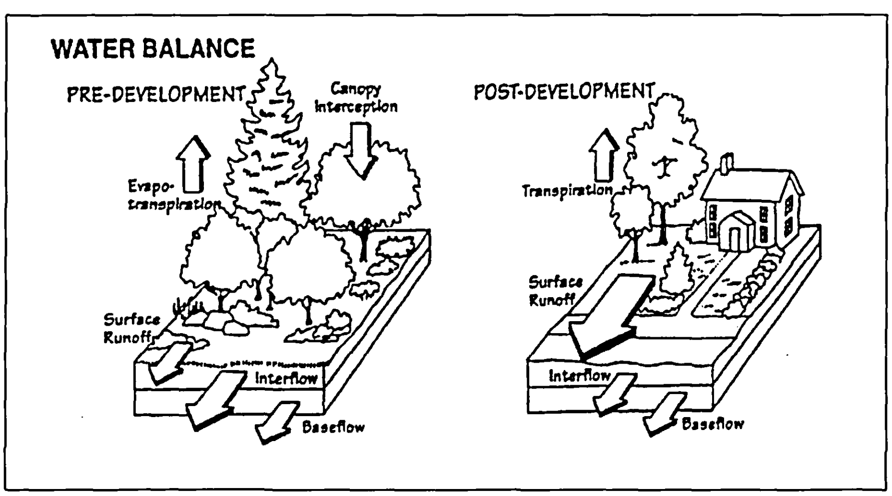



**Figure 2-1: Water balance at an undeveloped and developed site (Schueler, 1987)**

Historically, cities have installed storm sewer systems, which quickly channel this runoff from roads and other impervious surfaces. Runoff gathers speed once it enters the storm drainage system.

When stormwater leaves the system and discharges into a receiving water body, large volumes of runoff may erode stream banks, damage streamside vegetation, and widen stream channels (see Plate 2-1). In turn, this will result in lower water depths during non-storm periods, higher than normal water levels during wet weather periods (Figure 2-3), increased sediment loads and higher water temperatures. Native fish and other aquatic life cannot survive in many urban streams severely impacted by urban runoff.

The following flowchart summarises the impact of urbanisation on the water balance.

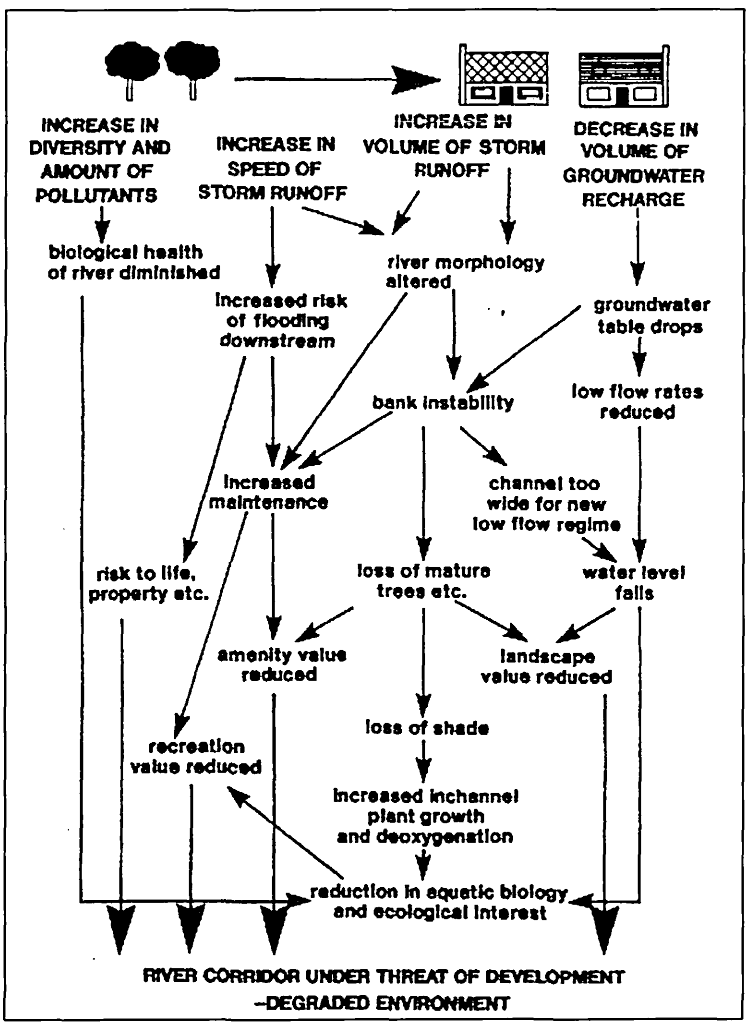



**Figure 2-2: Possible impacts of urbanisation via hydrological change (Gardiner, 1991)**

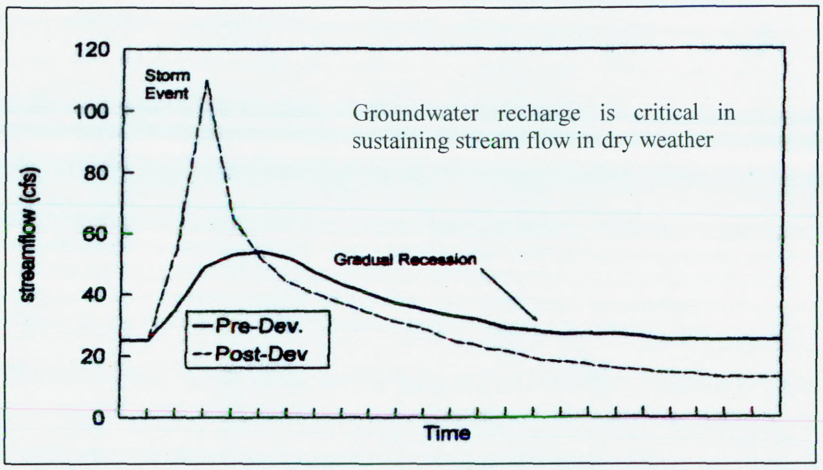



**Figure 2-3: Decline in stream flow due to diminished groundwater recharge (Center for Watershed Protection, 1997)**

**Plate 2-1: Stream bank erosion of the Lyne Burn in Eastern Scotland**

## 2.2 Urbanisation and pollutant loads

Urbanisation also increases the variety and amount of pollutants transported to receiving waters. Sediment from development and new construction; oil, grease, and toxic chemicals from automobiles; nutrients and pesticides from turf management and gardening; viruses and bacteria from failing septic systems; road salts; and heavy metals are all examples of pollutants generated in urban areas.

Vehicular traffic is directly responsible for the deposition of substantial amounts of pollutants, including toxic hydrocarbons, metals, and asbestos, in addition to oils. The particulates contributed by traffic are primarily inorganic. However, vehicle exhaust pipes are not the only source of traffic-related pollution. Tyre-wear, solids carried on tyres and vehicle bodies, and loss of fluids add to the pollution inputs contributed by traffic (Novotny and Olem, 1994). Increased pollutant loads can harm fish and wildlife populations, kill native vegetation, contaminate drinking water supplies, and make recreational areas unsafe.

## 2.3 An overview of SUDS in Scotland

In 1994 the former Forth River Purification Board (FRPB) completed a review of the causes of the poor quality reaches of the rivers in its areas. That review identified non-point or diffuse pollution, including urban runoff, as a significant issue (D'Arcy and Roesner, 1997). Pollution prevention and abatement concepts, such as Sustainable Urban Drainage Systems (SUDS) were introduced in a guidance document produced by the former FRPB and this was reproduced by SEPA (2000). SEPA identified three main objectives for SUDS, schemes as summarised below. Figure 2-4 illustrates the general approach to pollution prevention.

- The prevention of damage to streams and rivers by holding urban surface water runoff at or near the source;
- The provision of additional nature conservation benefits;
- The provision of additional amenity benefits.

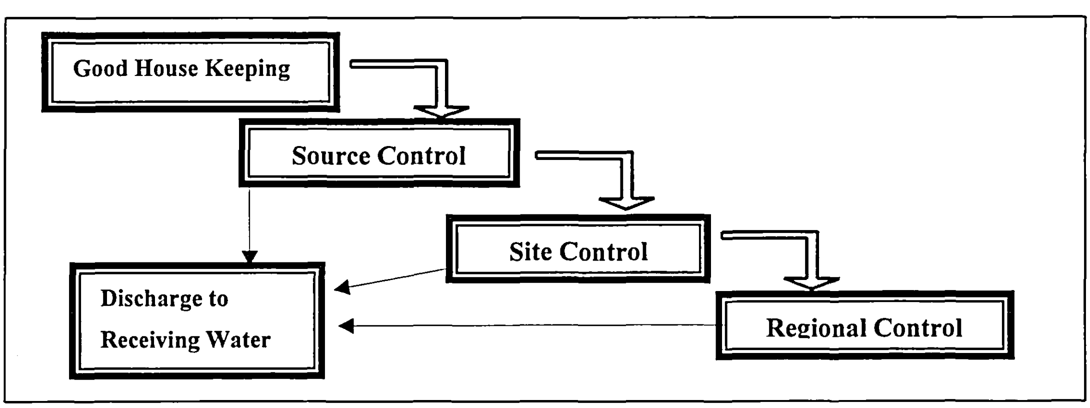



**Figure 2-4: The stormwater management train (adapted from CIRIA, 2000a & b)**

The stormwater management train is as follows; good housekeeping to reduce accidental or careless contamination; source controls which attenuate runoff while removing pollutants close to their origin; site controls providing local detention or treatment; and regional controls including end-of-pipe wetlands or ponds where necessary.

There is increasing pressure in the UK by the Environment Agency (EA) and the Scottish Environmental Protection Agency (SEPA) to consider the use of SUDS as part of standard drainage design (Kellagher, 2000). National planning guidance for Scotland stipulates that all planning applications for new developments should incorporate drainage strategies and designs, including proposals for Sustainable Urban Drainage Systems (Scottish Executive, 2001). Construction companies, developers and designers are being encouraged to adopt a sustainable approach to drainage design. Table 2-1 gives a brief description of the SUDS types applied in Scotland.

| **SUDS Type** | **Description** |
|---|---|
| Detention Basin | Storage facility to detain water. Hydrographs are attenuated |
| Retention Pond | Storage facility which retains water long-term. |
| Infiltration Basin | Similar to pond but all water stored is exfiltrated from the basin into the underlying soil. Surface-based structure |
| Wetland | Pond with purifying plants |
| Swale | Linear grassed depression allowing limited amount of storage, used for flow conveyance and possibly infiltration |
| Infiltration Trench | Trench filled with media having large void ratio allowing water storage underground |
| Porous Surfaces | High porosity pavements, generally car parks, with storage below surface. This type of system can either infiltrate or attenuate flows |
| Filter Strips | Grassed area for stormwater overflow allowing sedimentation prior entry to another SUD |
| Filter Drain | Perforated pipe in gravel surround allowing exfiltration but attenuates forward flow |

**Table 2-1: Description of SUDS (modified from Jefferies, 1999)**

SUDS have been promoted for a number of years in Scotland in response to the need to combat pollution arising from diffuse sources in urban areas and to promote flow attenuation (Schlüter et al., 2002). A study undertaken by Wild et al. (2002) showed a rapid increase in the number of SUDS since 1996 with a total of almost 4000 SUD systems on 767 sites around Scotland. It also showed that infiltration techniques were amongst the most popular, with almost 500 filter drains and infiltration trenches. Figure 2-5 shows the increase in number of SUDS sites, extrapolated to 2006 and Figure 2-6 shows the distribution of SUDS sites in Scotland.

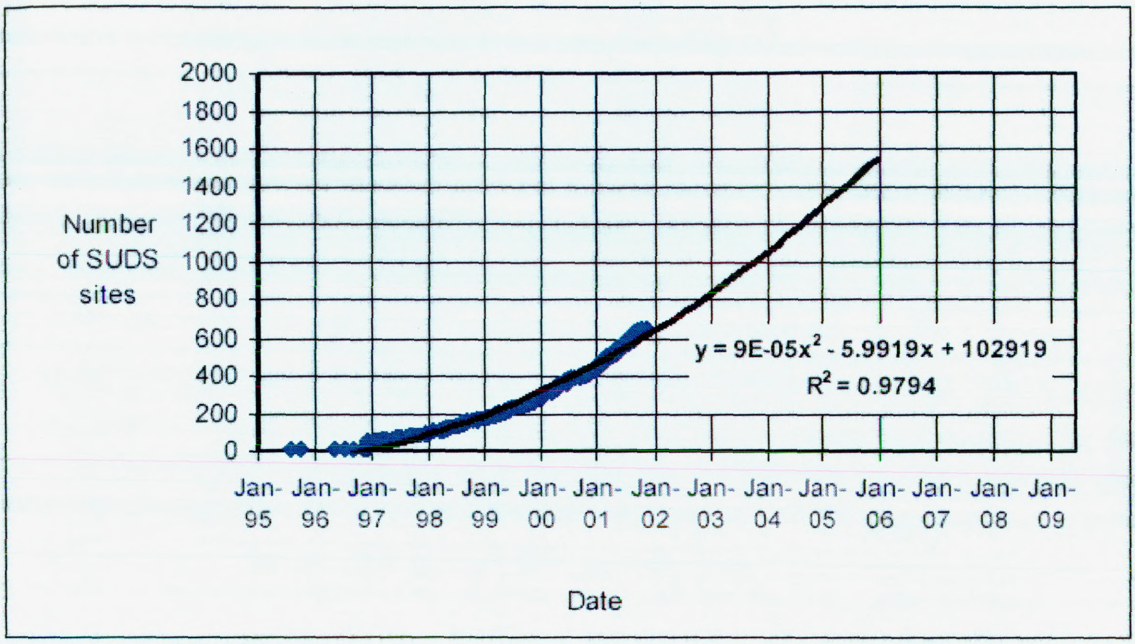



**Figure 2-5: Increase in numbers of SUDS sites in Scotland since 1995, extrapolated to 2006 (Wild et al., 2002)**

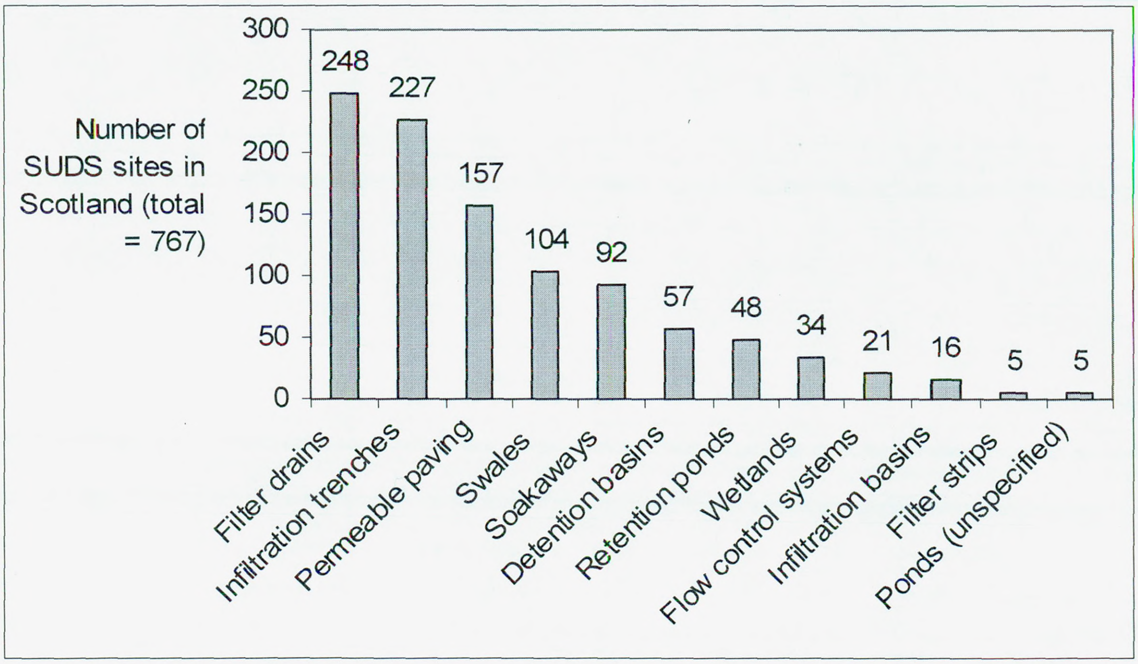



**Figure 2-6: Numbers of SUDS sites in Scotland (01/01/2002) (Wild et al., 2002)**

## 2.4 Review on infiltration trenches/ soakaways

Sustainable Urban Drainage System as outlined in Table 2-1 have been promoted as tools to mitigate the adverse effects of urbanisation and infiltration trenches, filter drains and soakaways are amongst the most popular SUD systems. This section provides a review of the current knowledge on these in-ground SUDS.

### 2.4.1 Description

Infiltration trenches are shallow excavations normally filled with stone to create temporary underground storage and infiltration of stormwater runoff. The runoff gradually exfiltrates through the bottom and/or sides of the trench into the subsoil and eventually reaches the water table. Figure 2-7 provides a schematic of a typical 'end-of pipe' system. Trench designs may be modified to include vegetative cover and other features, establishing a biofiltration area.

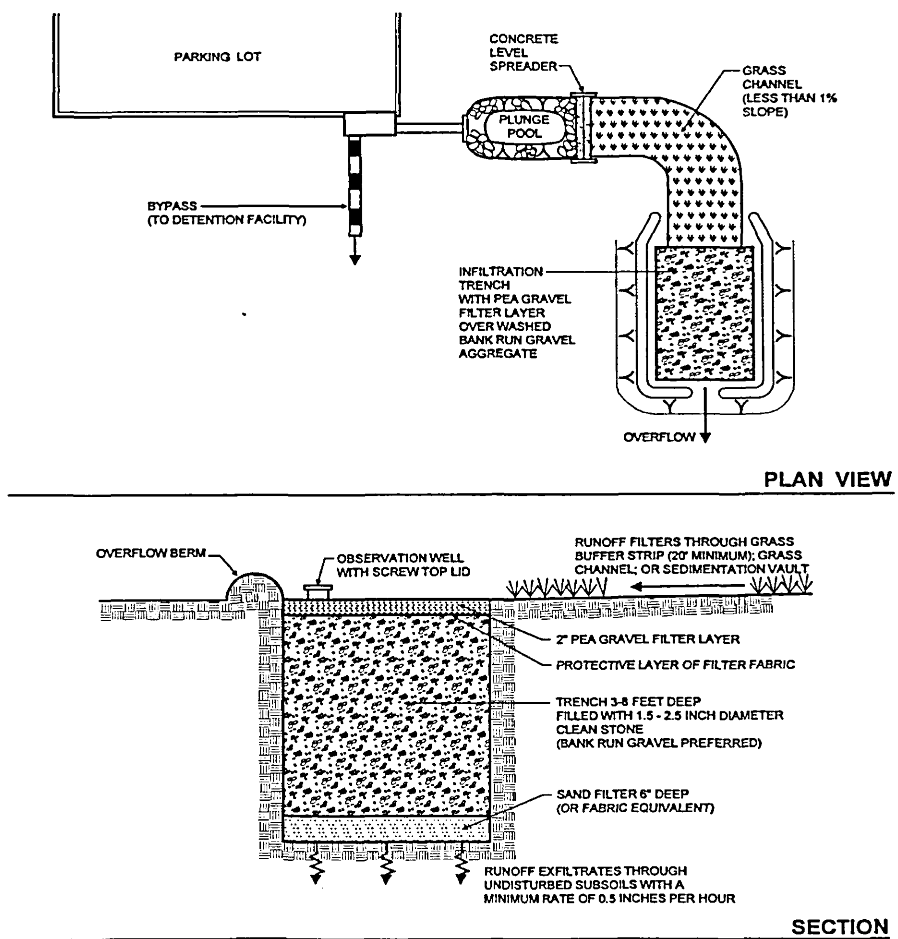



**Figure 2-7: Schematic of an infiltration trench system – US example (MDE, 2000)**

Infiltration trenches may be designed for either complete or partial exfiltration where the first flush volume is routed to the trench and the remainder is by-passed and conveyed to additional BMPs.

Infiltration trenches should always be constructed with pretreatment. The use of infiltration technologies should be avoided in high potential pollutant loading areas. In groundwater drinking supply recharge areas (Zone II and Interim Wellhead Protection Areas) infiltration technologies may be used for uncontaminated rooftop runoff only (Stormwater Management, 1997).

Soakaways are frequently used in rural areas for roof and road drainage and may have different forms and sizes. They are built as square or circular pits, which are filled with rubble or lined with dry-joined brickwork or pre-cast perforated concrete ring units surrounded by granular backfill. They can also take the form of trenches which follow convenient contours (BRE, 1991). Extensive use has traditionally been made of soakaways draining small catchments in remote and rural areas.

Filtration trenches or filter trenches, as they are commonly known, are often found in low permeability soils, where they attenuate flows, provide natural levels of soil moisture, and aid in purifying water prior to conveying it downstream. Perforated pipes buried within the gravel of a filter trench are often used to distribute the inflow runoff along the length of the trench (Duchene and McBean, 1992). Figure 2-8 shows a typical arrangement of this application.

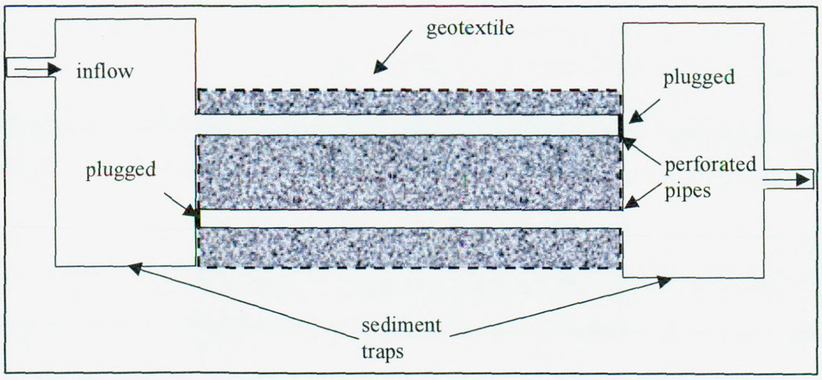



**Figure 2-8: Schematic of filter trench using perforated pipes as 'end-of pipe' system**

In roadside application drainage a perforated pipe in combination with an infiltration /filtration trench is known as a filter drain. The usual arrangement is in combination with a trapped gully pot. Figure 2-9 shows a schematic of a typical road drainage system. Investigation of a system with this arrangement showed unsatisfactory long-term performance (see Section 3.3.1)

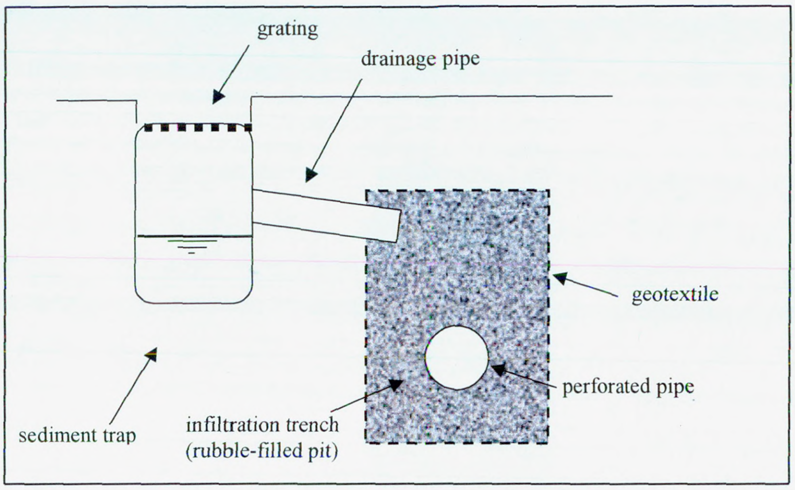



**Figure 2-9: Schematic of a filter drain for road drainage (modified from Price, 1994)**

It is estimated that filter drains are used to collect surface water runoff from 25% of all major roads in Great Britain (CIRIA, 1999). Despite the extensive use that has been made of infiltration pits and trenches or soakaways, there has been only limited examination of their performance (Warnaars et al., 1999). Abbott and Comino-Mateos (2001) state that lack of performance data is the main reason for the slow adoption of soakaways in the UK. There has been widespread concern about the hydraulic performance of these devices, with the general expectation that failure through blockage by silts and debris would necessitate reconstruction within a limited time period (Pratt, 2001 and Abbott and Comino-Mateos, 2001).

## 2.4.2 Performance

Infiltration trenches function similarly to rapid infiltration systems used in wastewater treatment. Estimated pollutant removal efficiencies derived from wastewater treatment performance and modelling studies are shown in Table 2-2. Based on this data, infiltration trenches can be expected to remove up to 90% of sediments, metals, coliform bacteria and organic matter, and up to 60% of phosphorus and nitrogen from the inflow (Schueler, 1992). Biochemical oxygen demand (BOD) removal is estimated to be between 70 to 80%. Lower removal rates for nitrate, chlorides and soluble metals should be expected for surface water runoff.

| Typical Percent Removal Rates | |
|---|---|
| Sediment | 90% |
| Total Phosphorous | 60% |
| Total Nitrogen | 60% |
| Metals | 90% |
| Bacteria | 90% |
| Organics | 90% |
| Biochemical Oxygen Demand | 75% |

**Table 2-2: Typical removal rates (Schueler, 1992).**

There is considerable variability in the effectiveness of infiltration trenches, and proper siting, design and maintenance improves their performance.

In the USA in the late 1990s research was undertaken on the water quality improvement capability of a partial infiltration trench at a site on the Millcreek Expressway in Cincinnati, Ohio (Sansalone et al., 1998 and Sansalone, 1999). Road runoff was intercepted by a small section of porous pavement and filtered through the granular backfilled trench. Water could infiltrate the adjacent soil or be discharged from the filter media via a pipe, the flow from which was monitored and sampled.
The heavy metal and total suspended solids removal efficiency results showed that the system was an effective trap. Field observations for dissolved metals showed removal efficiencies for Zn >95%; Cu >85%; Cd >80%; and Pb varying between 70-95%. The equivalent results for the particulate-bound metals were Cu and Pb 85-95%; Zn 75-95%; and Cd 79-90%. Concern was expressed about the potential for clogging of the system and the effect this would have on the design life of 10 years.

### 2.4.3 Limitations

Although infiltration trenches can be a useful management practice, they have several limitations, including the following (USEPA, 1997):

- High failure rate due to poor maintenance, wrong siting or high debris input.
- May not be appropriate for industrial sites or locations where spills may occur.
- Infiltration trenches are limited to relatively small catchments.

- Infiltration trenches and soakaways require a minimum soil infiltration rate or maximum emptying time depending on the location. This is not the case for filter drains of filter trenches.
- Their use is restricted at locations with high groundwater level and close to building foundations.
- Not suitable on fill sites or steep slopes.
- Risk of groundwater contamination.
- Upstream drainage area must be completely stabilised before construction.
- Cost of replacing filter material once blocked.
- Does not provide visual enhancement.

### 2.4.4 Siting criteria

The use of infiltration trenches may be limited by a number of factors, including type of native soils, climate, and high groundwater table. Site characteristics, such as excessive slope of the drainage area, fine-grained soil types, location of the water table and bedrock may all restrict the use of infiltration trenches. Generally, infiltration trenches are not suitable for areas with relatively impermeable soils containing clay and silt or in areas with fill.

However, in Sweden infiltration trenches have been successfully implemented in soils of a boulder clay of impermeable nature. They were found to have the benefit of maintaining an adequate level of soil moisture, preventing consolidation of the soil leading to unacceptable settlement of buildings. Holmstrand (1984) reported that up to two-thirds of the water discharged to the trenches was not passed to the outfall and that losses were due to evaporation from the soil and plant transpiration. Schlüter and Jefferies (2001) reported similar findings with 50% flow volume losses from a study on a sealed pervious pavement.

The potential for groundwater contamination must be carefully considered especially if the groundwater is used for human consumption or agricultural purposes. Serious concern has arisen with regard to groundwater contamination from highway runoff draining into filter trenches (Price, 1994). In Germany highway runoff is generally not permitted to be directly collected in infiltration trenches due to the risk of groundwater contamination (ATV 138, 2002).

The infiltration trench is not suitable for sites where hazardous spillages are likely to occur, such as industrial sites etc. In these areas, other BMPs, which do not allow interaction with the groundwater, should be considered.

Soil infiltration rates and the water table depth should be evaluated to ensure that conditions are satisfactory for proper operation of an infiltration trench.

Pretreatment structures, such as a vegetated buffer strip or water quality inlet, can increase longevity by removing sediments, hydrocarbons, and other materials, which may clog the trench. In the UK typical trapped gully pots are used in the application of road drainage and extensive use has been made of catchbasins.

### 2.4.5 Design guidelines

UK design guidelines are outlined in the BRE 365 (BRE, 1991), CIRIA reports C521 and C522 (CIRIA, 2002 a & b) and SNIFFER report No.SR(02)51 (Jefferies, 2003). German guidelines are in the ATV 138 (2002) and various guidelines have been published in the US: Schueler (1987), Stormwater Management (1997), USEPA (1999), MDE (2000).
Specific designs vary considerably, depending on site constraints or the preferences of the designer or community. However, there are some features which should be incorporated into most infiltration trench designs and these are discussed below:

- Infiltration trenches should always be constructed with pretreatment to reduce the sediment load. BRE 365 (BRE, 1991) suggests the use of wet sumps and T-piece inlets to distributor pipes in trench systems.
- Specify locally available granular fill material, e.g. trench rock that is 5 to 25 mm in diameter.
- Determine the trench volume. This depends on whether the trench is designed to treat the first flush or the total runoff volume. The volume defined is assumed to fill the void space based on the computed porosity of the rock matrix.
- Determine the effective area. In the US this is defined as the bottom surface area needed to drain the trench within 72 hr by dividing the treatment volume by the

infiltration rate. In the UK this is calculated using 50% of the total depth, excluding the bottom area. The trench should be half-drained within 24 hr.

- The sides and the bottom of the trench should be lined with permeable, geotextile fabric. A layer of fine sand (clean, fine aggregate) may be substituted or used in addition on the bottom.

- Provide observation well to allow observation of drain time.

Monster and Leeflang (1996) investigated design procedures of various countries and outlined several weaknesses. These include that the effective infiltration area is computed as constant although it is a time dependent variable and that design procedures presume that the device is empty at the start of an event. These authors then proposed a more realistic approach that uses a Storage-Design Discharge-Frequency curve based on 14-year time series rain data. Using this approach allows for a dynamic percolation area and takes account of filling variations during subsequent rainfall events.

Each country has developed its own design procedure for infiltration facilities. Moreover, the German and Swiss directives show design procedures for each individual system. The Dutch directive contains a general procedure, which is suitable for all systems. Table 2-3 shows the recommended return periods used in each country (Monster and Leeflang, 1996).

| Country | Denmark | Germany | Holland | Sweden | Switzerland | UK | USA |
|---|---|---|---|---|---|---|---|
| T (years) | 2 | 5 | Choice | 2 | Choice | 10 | Varies |

**Table 2-3: Recommended return periods (Monster and Leeflang, 1996)**

Each country has developed a Storage-Design Discharge-Frequency curve for the design of the infiltration facility. The Netherlands, Switzerland and Denmark developed these curves using a rain series, whereas the other countries used a rainfall Intensity-Duration-Frequency curve.

Runoff into a conveyance system taking losses into account is only described in the directives of Sweden and the USA. In the Netherlands there is a choice between curves based on rainfall or inflow.

The soil permeability is often difficult to assess. Some countries use data tables for each soil type, other countries require on-site investigations and the Dutch procedure allows both.

Another parameter of great diversity is the effective percolation area. In many countries the bottom of the facility is assumed to be clogged after some time and is therefore not taken into consideration. Most countries use 50% of the side walls. Although this is a time dependent variable, it is used as a constant for the design approach. None of the procedures acknowledge the change of head (Monster and Leeflang, 1996).

Duchene and McBean (1992) undertook modelling on the infiltration characteristics using a two-dimensional saturated-unsaturated finite model. Results show that approximately three quarters of runoff infiltrate through the bottom and that the impact of bottom clogging is important but limited. Even with a 50 mm-layer of clay along the bottom, water still infiltrates through the bottom of the trench.

Todorovic et al. (2002) developed a model which incorporates a water flow model and a pollutant flux model. Simulation results show that neglecting problems of clogging in design could lead to serious malfunction of their performance, causing frequent overflow.

Abbott and Comino-Mateos (2001) undertook testing of UK design guidelines on a soakaway system and concluded a reasonable match of computed and monitored maximum water level within the 2-year on-site observation. Although emptying times of up to 3 weeks were recorded, no overflow occurred. This indicates that such systems can function satisfactorily with much longer emptying periods.

Infiltration practices have historically had a high rate of failure. One study conducted in Prince George's County, Maryland (Galli, 1992), showed that less than half of the infiltration trenches investigated (of about 50) were still functioning properly, and less than one-third still functioned properly after five years. Many of these practices, however, did not incorporate advanced pretreatment. By carefully selecting the location and improving the design features of infiltration practices, their performance should improve (USEPA, 1999).

### 2.4.6 Maintenance

Longevity can be increased by careful geotechnical evaluation prior to construction and by designing and implementing an inspection and maintenance plan.

Observations from Tokyo, Japan (Haneda et al., 1996) suggest that such systems may operate satisfactorily with significant benefits in the reduction of direct discharges to watercourses. A key factor in continued satisfactory operation is the design, operation and maintenance of inlet structures to the underground devices. Where roof water discharges into sediment/debris traps, the underground, stone-filled trenches retained their 'infiltration capacity as it had been at the beginning after 11 years of service'. However, where the waters entered the system from paved surfaces, carrying considerably more silt and debris, the infiltration rates fell rapidly due to blockage (Pratt, 2001).

This confirmed the previous findings by Minagawa (1990). Excavations of infiltration trenches revealed less silt within the stone fill and on the base of the trenches than expected. Extensive use was made on site of U-shaped collecting channels and of silt traps: these played an important role in the pre-trapping of materials before entry to the infiltration trenches.

Price (1994) points out that gully pots in the UK were formerly emptied every two months. Following reorganisation of the UK local government in 1974, the interval between gully emptying has increased to a year or more. Data published in CIRIA Report 134 (CIRIA, 1995) shows gully pot sediment filling times of approximately 4 to 6 months. There is concern that water held in traps for a long time may provide a breeding ground for bacteria and concentrate pollutants which are then carried out as a concentrated 'first flush' when it rains (CIRIA 1999, Price 1994, Butler and Memon, 1999). In the US trapped gully pots are not used as they may provide a breeding ground for mosquitoes.
Anecdotal information from key personnel at the water authority and highway operators revealed that maintenance in the East of Scotland is undertaken mainly on an incident basis. The reasons given were shortages of staff and infrastructure limitations.

The following points should be considered (USEPA, 1999):

- Observe drain time for the design storm after completion or modification of the facility to confirm that the desired drain time has been obtained (BRE, 1991).

- A detailed maintenance programme with inspection schedule is vital to ensure longevity of the filtration /infiltration device. The schedule should vary with location and system used.

- Inspections shall be undertaken at each manhole and may involve manhole entry. Maintenance involves the emptying out of the manholes as well as the distributor pipes.

- Remove accumulated trash and debris in the system as often as practicable.

- If sediment is visible on top of the trench, remove top layer of trench, silt, filter fabric and stone; wash stone and reinstall fabric and stone into trench.

- Inspect for standing water after main rainfall events.

## 2.5 Overview of computer simulation in urban drainage

The modern approach to representing the rainfall-runoff process on urban catchments is over a hundred years old. The Bürkli-Ziegler (1880) formula was presented in 1880 and Kuichling (1889) published the rational formula in 1889. These formulas estimated only the peak discharge of storm runoff, mainly for sizing drainage pipes. Subsequently, many methods were developed for the purpose of providing additional information on the various processes. These methods have often been referred to as models. The term "model" can be defined as a method to represent mathematically, a physical process allowing different values of the input parameters (boundary and initial conditions) to generate different values of outputs (Maksimovic and Radojkovic, 1986).

In general, drainage models describe the transformation of the input (rainfall) through the catchment (runoff) and sewer network to produce the output at a point of interest (outlet of catchment). Figure 2-10 shows the scheme of a standard urban drainage model.

| | |
|---|---|
| Precipitation | (design storm, historical storms) |
| **Loss model** | (wetting, evaporation, infiltration, storage) |
| Net precipitation | (effective rainfall) |
| **Surface runoff model** | (surface definition, transport process) |
| Sewer inflow | (type of inlet, head losses) |
| **Pipe flow model** | (storage, transport process) |

**Figure 2-10: Schematic of an urban drainage model (Smart and Herbertson, 1992)**

In the mid-1970s computer modelling became an integral part of storm drainage planning analysis and design. The rapid development of microcomputers in the 1980s made it possible for virtually every drainage engineer and planner to use analytical methods for purposes ranging from analysis of individual pipes to comprehensive stormwater management plans of entire cities (The Wallingford Procedure (1981a), American Society of Civil Engineers (1994)). One of the most popular models which appeared in the US in the early 1970s and has continued to develop ever since is the Storm Water Management Model (SWMM) (Computational Hydraulics, 2004). In the UK, the Wallingford Procedure (WASSP), was introduced as first standard software package in the early 1980s. The latest standard modelling packages are SWMM, InfoWorks, Mouse, WinDap, etc and these are based on accepted mathematical relationships between the physical parameters.

## 2.5.1 Computer simulation programs

This section gives brief descriptions of the computer programs which were available for modelling the performance of the in-ground SUDS and these are as follows:

- The storm water modelling program Erwin
- WinDes/WinDap
- Infoworks
- Hydrol Times Series Manager

Simulation provides an in-depth understanding of the system being investigated. The approach is somewhat different from drainage design. Usually, a monitoring program is undertaken for several months, providing a high resolution of measurement points. The existing system is then represented using mathematical software and a sensitivity analysis is undertaken prior to the calibration and verification process.

The sensitivity analysis shows the influences of the various factors affecting the system's performance. Once a model is developed, performance comparisons between various system elements can be undertaken easily and the system's long-term behaviour may be predicted.

Today, a main challenge is integrated stormwater modelling, which consist of investigating the interactions between sewer network, groundwater, overland and river flow-models. Finding solutions for the ever-increasing flood incidences is one of the most pressing applications of integrated stormwater modelling, and the hydraulic influence of in-ground SUDS may play an important role in such systems. Although different flow networks can be simulated separately using specialised software, the linkage of different systems and different software packages is limited. To date, no comprehensive modelling package is readily available covering all components of the different drainage systems. Flow simulation through SUDS elements is not supported or represented by use of simple 'black-box' models, which are not very accurate and cannot be linked to standard modelling packages. The flow simulation from these 'black-box' models is based upon a head-discharge relationship, which has to be specified by the user. Often, lack of information, time and financial restrictions result in a rough approximation of the head-discharge relationship, which may produce false simulation results. Flow characteristics based upon the physical characteristics of the systems are desirable.

Infoworks is one of most popular software packages for catchbasin as well as sewer-network modelling but no SUDS elements are represented, to date. To enable Infoworks to maintain its leading role in drainage design software, it must incorporate SUDS capabilities. This will become an important selling feature in the near future, and Infoworks sales will suffer without such capabilities (Kellagher, 2000).

### 2.5.2 The storm water modelling program Erwin

Erwin is an icon driven rainfall-runoff model for urban drainage which contains all the modules needed for designing and evaluating the performance of sustainable urban drainage systems (Abertay Waste Solutions Ltd, 1998). The system is built graphically on the screen.

Key features are:

- graphical on-screen system design
- interactive initial design to UK and European standards
- long-term simulation
- single event simulation with on-line visualisation
- direct transfer of output into project reports
- parameter input by mouse for all system components.

All the components of SUDS-based and conventional urban storm water management are supported: pervious and impervious areas; storm tanks; central and distributed infiltration systems; trench soakaways, ponds; swales, channels and pipes.

Simulation results are clearly presented as hydrographs and flow balances on screen. System data, results and diagrams can be directly transferred to the Windows software.

The basic model used in Erwin to calculate the runoff process from impervious surfaces is the maximum rating method by Paulsen (Abertay Waste Solutions Ltd, 1998). This method allows for a continuous calculation of depression filling and emptying.

The runoff from pervious surface sub-models describes the process with the help of the maximum rating method derived by Paulsen and Horton's infiltration method.

The infiltration capacity of the ground depends on the permeability of the soil and on the actual water content, which may be variable. Infiltration of rainfall into the ground is calculated according to the Horton/Paulsen approach. The Horton approach formulates infiltration as a function of the time between an initial and a final infiltration rate. The basic assumption is that the rainfall intensity is always greater than or equal to the actual infiltration rate. The Paulsen modification extends the range of validity for the Horton approach to phases with rainfall intensities less than the actual potential infiltration. Further, the return to the initial infiltration rate, during phases with little or no rainfall, can be calculated. A detailed description of the parameters can be found in the Software Manual (Abertay Waste Solutions Ltd, 1998).

### 2.5.3 WinDes/WinDap

WinDes is a software tool to design drainage systems. It includes features for dimensioning traditional pipe network systems as well as a number of source control devices. These include storage tanks and ponds, infiltration basins, swales, porous pavements, soakaways, infiltration trenches. SUDS elements are often employed in combination with flow control devices of which the following are provided: Orifices, weirs, flumes, gates, pipes, vortex flow controls, user-definable depth/flow, pumps, plane and 3 dimensional infiltration.

WinDes uses a rainfall generator based on the modified rational method and calculates the dimensions for the system specified. Graphical features provide for longitudinal or cross-sectional view as well as for water flow animation. In addition, it is compatible with AutoCAD and makes use of electronic location maps for in-plan design applications. It also provides design for a network of the SUDS features and calculates the downstream flow behaviour.

A feasibility tool is provided to enable cost and benefit comparisons of employing storage with and without infiltration.

### 2.5.4 Hydrol TSM

Hydrol Time Series Manager (TSM, 2000) provides two different programs; a database with various tools for data analyses in addition to a modelling package. The modelling package is a node-link-based program which allows access to data directly from a database and provides a number of simulation scripts (text file containing codes) to perform hydraulic simulation. Hydrol allows modification of these scripts for individual conditions and the main scripts are listed here:

- Percentage loss model
- Initial and continuous loss model
- Time delay
- Catchment routing
- Channel routing

In addition to these scripts the program allows the users to write their own scripts, which makes the program extremely flexible. No SUDS are applicable to date and users would be required to develop a specific script to represent SUDS. The modelling package is based on a language similar to visual basic, which is called Hydrol Basic and this incorporates all standard conditional expressions, i.e.: IF THEN ELSE, DO UNTIL, DO WHILE, etc.

This program has a graphical output, and simulation can be undertaken in continuous or step mode. The step mode is useful for error checking of every single model component.

The software is designed for handling time series data, and input and output to the program is continuous data in a specific time interval. If the data differs from the set time interval the program performs interpolation or aggregation.

### 2.5.5 Infoworks

Infoworks is a modelling program, which incorporates the key elements of wastewater and combined sewer systems. It supports modelling of backwater effects and reverse flow, open channels, trunk sewers, complex pipe connections and complex ancillary structures. InfoWorks provides interactive views of data using plan views, long sections, spreadsheet and time varying graphical data. The Water Quality Module can model physical processes such as the first foul flush, sediment built up behind closed gates and penstocks. Using the Water Quality Module, engineers can control pollution by targeting the SSO and CSO problems, and predict quality components such as the volume of spillage and flooding (Wallingford, 2000). To date, a soakaway manhole is the only SUD element available in Infoworks.

## *2.6 Risk Assessment Methodologies*

A short introduction to strategies for environmental risk assessments is provided here to give an overview of the various methodologies which are in use. Risk assessments have been used for various applications and can be classed as qualitative, semi-qualitative and quantitative. A useful guide to the selection of methods is given by Pollard et al. (1995).

*Qualitative Methods*

Qualitative methods have been used for screening risk (Pollard et al., 1995) and for the assessment of financial liabilities (Pritchard, 2000). Both strategies involve hazards-pathways-receptors analyses.

*Semi-Qualitative Methods*

Semi-qualitative methods involve ranking risks according to a predefined risk matrix which takes into account the degree of hazard and the effects of each risk. Inherent in these methods is the categorisation of risks according to what can be termed risk exposure, to allow a more detailed examination and analysis of the most important risks. The main categories of risk exposure can be defined as:

- High probability - High impact
- Low probability - High impact
- High probability - Low impact

- Low probability - Low impact

In general terms, risk can be defined as the probability of an event multiplied by the magnitude of the loss. Environmental risks therefore arise due to the impact of a hazard. Risk assessment requires, firstly, the identification of the presence of a hazard and, secondly, an assessment of its impact. From the above definition, Environmental Risk can be assessed in terms of the probability of damage to something of human or ecological value arising from an exposure to a hazard. This impact can be either intense, where severe consequences occur in a very short time-scale, or diffuse, occurring over a medium to long term time period.

Pritchard (2000) describes examples of semi-qualitative methods. In these methods a group of "experts" is assembled to assign elements to a ranking matrix based on severity and probability. Individual matrix cells give an indication of the priority to be attached to the hazard. The matrix and the procedure for completion and assessment of the risk arising from industrial fires are shown in Figure 2-11 as an example.

| | | Severity: Very Severe 1 | 2 | 3 | 4 | Low Consequences 5 |
|---|---|---|---|---|---|---|
| Probability: High | 1 | 1 | 2 | 3 | 5 | 7 |
| | 2 | 2 | 4 | 6 | 7 | 8 |
| | 3 | 3 | 5 | 7 | 8 | 9 |
| | 4 | 4 | 6 | 8 | 9 | 10 |
| Low | 5 | 5 | 7 | 9 | 10 | 10 |

Severity factors:
1 Fatality
2 Severe injury
3 Incident resulting in major clean-up
4 Minor injury as result of exposure
5 Minor injury, such as cuts, trips, falls

Probability factors:
1 Once per year: High
2 Once per 10 years: Moderate
3 Once per 100 years: Medium
4 Once per 1,000 years: Low
5 Once per 10,000 years: Very low

**Figure 2-11: Risk Ranking Approach (Pritchard, 2000)**

In the context of urban drainage, typical examples of severity factors would be property flooding, road flooding, water quality pollution due to accidental spillages, ground water pollution, etc. The probability factors are dependent on the specific risk assessment and with regards to in-ground SUDS this would range from bi -annually up to the system's design life, e.g. once per 10 years: The procedure involves gaining an understanding of the processes involved in the project; all the imagined hazards arising should be listed. They are categorised by Severity and Probability and then plotted on a priority chart as shown above (e.g. Severity 1 Probability 3 would be priority 3).

### *2.1.3 Quantitative Methods*

Quantitative risk analysis attempts to express risks mathematically by modelling the exposure to sources by aggregating the exposures over all routes and sources and expressing estimated risks to all receptors or groups of receptors.

Such an analysis requires the combination of a number of mathematical modelling techniques to assess the exposure level of the recipients at the end of the pathways and the application of toxicological assessment methods to assess the effects on receptors (Hughes, 1996; Rand, 1995).

## 2.7 Knowledge enhancement

A rapid growth in the installation of SUDS since 1996 with a total of 4000 SUD systems and just under 800 sites around Scotland was reported by Wild et al. (2002). The study also showed that infiltration techniques were among the most common with almost 500 filter drains and infiltration trenches to date and this is expected to rise to over 1200 in-ground SUDS by 2008.

Despite the popularity of in-ground SUDS, there is a lack of information on the optimal detailing, operation and maintenance of in-ground SUDS that would contribute to a satisfactory long-term performance. Differences in system design, climate and catchment characteristics limit the direct transfer of findings from other countries and UK based research was urgently needed. The following sections outline the gaps in current knowledge and show how this study contributes to the enhancement of knowledge in the field of in-ground SUDS.

### 2.7.1 Detailing

Very limited information exists on the detailed design of in-ground SUDS. Post construction checks are generally not carried out and there is some concern about the

performance and environmental impact of systems. Section 2.4.5 describes a number of design guidelines available. Unfortunately, these design guidelines mainly provide information for sizing the various systems. Very limited information exists on the detailed design of in-ground SUDS and this is of great importance for satisfactory performance.
This study enhances knowledge on the detailing of infiltration trenches and filter drains. This has been achieved by undertaking different investigations and analyses, which were as follows:

- On-site monitoring of six typical systems to gain in-depth knowledge of hydraulic behaviour.
- Development of generic models to aid the hydraulic analyses.
- Visual inspection and review of design information of 43 systems to reflect on the vast number of systems installed.
- Introduction of the Schlüter scoring system to enable a direct comparison of performance between all systems.

This approach was carried out to obtain an in-depth understanding of typical systems and also to reflect on the general performance of the vast number of systems installed to date. The experience which was gained from the hydraulic investigation enabled a good approximation to the likelihood of performance to be made for the remaining systems which were reviewed. The outcome from this part of the research is a set of characteristics and features which provide for satisfactory operation and also specify elements and characteristics which were found to result in the failure of in-ground SUDS (see Sections 4.5 and 4.6).

### 2.7.2 Maintenance

To date, no study has been undertaken in the UK of appropriate maintenance operations for filter drain and infiltration trench systems, and current maintenance techniques are still based upon traditional drainage systems. There are generally no routine maintenance programmes in place in the East of Scotland and maintenance is undertaken on an incident basis. Better information on maintenance techniques and intervals to enable a satisfactory operation of in-ground SUDS is urgently needed and this study provides an inventory of short and long term maintenance requirements. Different approaches were taken which led to knowledge enhancement on maintenance and these are as follows:

- On-site review of current maintenance techniques.

- Investigation of maintenance effects on water quality and hydraulic performance.
- Anecdotal evidence from highway operators and water authorities with regards to maintenance intervals and programmes.

In addition, the maintenance operations that was undertaken were critically appraised and operational improvements are detailed in Section 4.8.4.

### 2.7.3 Hydraulic performance

The information on the performance of infiltration trenches and filter drains shown in Section 2.4.2 is based on studies undertaken in the US. There have been very few studies undertaken in the UK and because of the recent popularity of in-ground SUDS, there is an increasing demand for up-to-date information on system performance. On-site monitoring of six filter drain and infiltration trench systems was undertaken to gain a detailed understanding of their hydraulic performances, including outflow volume, peak flow reduction and initial runoff loss. In addition, a comparison of the different sites was carried out, which allowed for an identification of the differences in hydraulic performance and enabled an evaluation of the treatment volume at each site. Design rainfall as well as recorded time series data were used for the model comparison.

### 2.7.4 Computer Simulation

The main software packages for hydraulic simulation of drainage systems in the UK are described in Section 2.5. The review of available packages in addition to discussions with personnel from leading software companies in the field of urban drainage, gave evidence that, to date, no standardised program was available that truly represents the flow characteristics of in-ground SUDS. A flow model based on Darcy's law using finite volume approximation was developed in collaboration with Wallingford Software and this model is proposed for incorporation in Release 6.5 of Infoworks. The initial model was developed using a spreadsheet, which was then translated to a code-based procedure. Model validation used recorded data from a study site at Walker Dam in Aberdeen. The advantage of the newly developed FVD model in comparison with existing models is that it is based on the system's physical characteristics, which will provide a realistic representation without the need of extensive data monitoring.

# CHAPTER 3 MONITORING OF IN-GROUND SUDS

*This chapter presents information from on-site monitoring of selected filter drain and infiltration trench systems. The knowledge and experience gained from on-site monitoring provides the focus of this research, which enables hydraulic analysis, performance comparisons, modelling, etc. Please refer to Appendix, B, C and D for more information. Appendix B gives more details on the monitoring sites, Appendix C provides detailed monitoring results including hydrographs, event tables and relationships, and construction drawings are included in Appendix D.*

## 3.1 Rationale

On-site monitoring was undertaken at six selected sites to gain a detailed understanding of the hydraulic behaviour of different infiltration trench and filter drain systems. In addition, water quality monitoring was undertaken for selected periods to assist in understanding the systems' pollution retention capabilities. Experience from on-site monitoring, in combination with the visual inspection-survey, enabled a better identification of good and bad detailing (see Section 4.5 and 4.6). Rainfall and outflow monitoring data were then used for developing individual models of each site allowing a direct performance comparison. The monitoring also provided data for model validation of the newly developed Finite Volume Darcy Flow (FVD) model.

## 3.2 Methodology

On-site flow monitoring was carried out in order to gain a detailed understanding of the hydraulic behaviour of in-ground SUDS. In addition, water quality was monitored during targeted events to gain an understanding of the systems' pollution retention capabilities.

The investigation was carried out at various levels to obtain both an in-depth understanding and to reflect the general performance of the large number of systems installed to date. It was decided therefore to undertake on-site monitoring of three filter drain and three infiltration trench systems.

One raingauge and two flow meters were installed at each site and water quality sondes were installed to record targeted events. Section 3.5 provides information on the instruments used for hydrological, hydraulic and water quality monitoring.

Flow was monitored at the inlet and the outlet of each infiltration trench system. Analyses were carried out for the following parameters:

- Percentage runoff from the drained area
- Flow volume loss at each system
- Peak flow reduction
- Lag time
- Antecedent precipitation

Flow was monitored at either one or two locations, allowing performance investigation of each corresponding filter drain section. Analyses were undertaken for the same parameters as for the infiltration trenches. Section 3.6 gives detailed information about the hydraulic analyses.

## 3.3 Site Overviews

Six sites were monitored in detail and these were located in the East of Scotland. This section gives an overview of the systems and Appendix B provides more detailed information about each system and detailed construction drawings are provided in Appendix D. Table 3-1 presents a data summary of the systems monitored and Figure 3-1 shows the monitoring locations.

| Site Name, Location | Type of SUD system | Monitoring | | Catchment Area | Pipe Diameter |
|---|---|---|---|---|---|
| | | Start | End | [m$^2$] | [mm] |
| Lang Stracht, Aberdeen | Filter Drain | Jan-00 | Oct-02 | 9520 | 225-300 |
| A90, Glencarse | Filter Drain | May-03 | Nov-03 | 2526 | 150-225 |
| Spine Rd, Dunfermline | Filter Drain | Mar-03 | Jun-03 | 3057 | 150-225 |
| Broxden, Perth | Infiltration Trench | Nov-02 | Feb-03 | 7500 | 150-225 |
| Walkerd Dam, Aberdeen | Infiltration Trench | Nov-02 | Feb-03 | 7000 | 150-225 |
| Transy Estate, Dunfermline | Infiltration Trench | Mar-03 | Jul-03 | 6568 | 150-225 |

**Table 3-1: Information overview of monitored systems.**

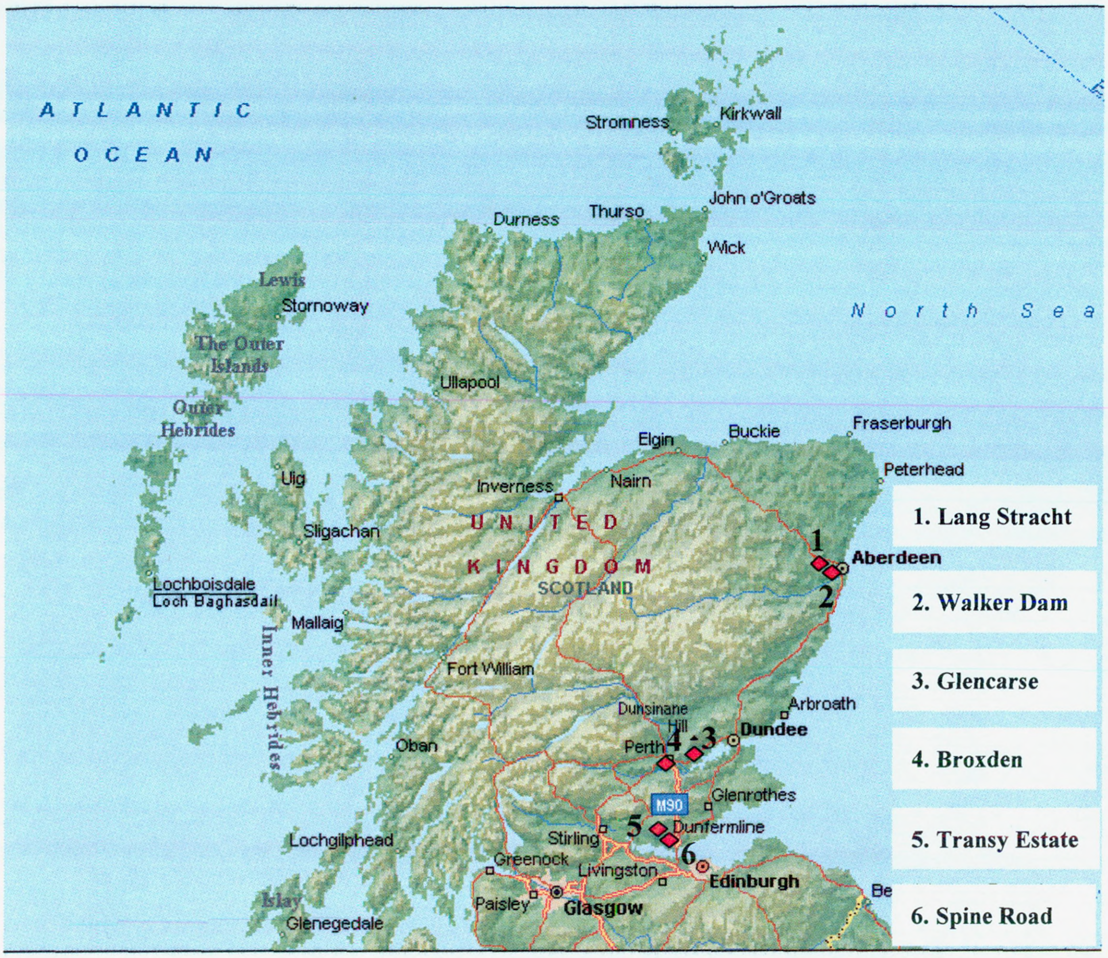



**Figure 3-1: Monitoring locations.**

### 3.3.1 Filter drain along Lang Stracht, Aberdeen

The filter drain system is located alongside Lang Stracht, a section of the A944 in Aberdeen. It consists of 750 metres road drainage, including 25 trapped gully pots and 10 catch pits. It is a typical kerb gutter system with trapped gully pots discharging into the filter material. The gully outlets are disconnected from the drainpipe, which is located 0.5m above the base. The filter drain is covered by a 150 mm layer of topsoil. Appendix D.1 shows a typical section through the filter drain. Road-runoff is attenuated and treated prior to discharge to the Denburn (see Appendix B.1 for more detail on site information and C.1 for detailed monitoring results).

### 3.3.2 Filter drain along Spine Road, Dunfermline

This filter drain is located alongside Spine Road, located within the Dunfermline East Expansion Area (DEX). It is a typical kerb gutter system with trapped gully pots discharging directly into the perforated pipe (see Appendix D.2). The drainage system includes 15 typical trapped gully pot inlets and 4 sediment chambers on a length of 255 metres. The perforated drainpipe is located approximately 0.2m above the base of the trench. The Gully pot outflow is distributed along the trench from within the perforated pipes. In addition, sheet flow enters the trench directly from the footpath. The filter drain system is part of a treatment train that conveys flow into a detention basin prior to discharge into the Linburn Pond (see Appendix B.2 for more detail on site information and C.2 for detailed monitoring results).

### 3.3.3 Filter drain along the A90 East of Glencarse

This filter-drain system is located alongside the A90 dual carriageway East of Perth, near to Glencarse. It is a typical road drainage system receiving lateral sheet flow from the dual carriageway, which is discharged into a local watercourse. The perforated drainpipe is located approximately 0.2m above the base of the trench (details of the filter drain are included in Appendix D.3.) Inspection chamber are located in 100 metre intervals (see Appendix B.3 for more detail on site information and C.3 for detailed monitoring results).

### 3.3.4 Infiltration trench at housing estate Walker Dam, Aberdeen

The system at Walker Dam is a typical end-of-pipe solution for a small housing estate. The perforated drainpipe is disconnected from the system inlet and located just above the base of the system. There are two inspection chambers, one upstream and one downstream of the trench and these are built as sediment traps (Appendix D.4 shows a detailed construction drawing of the system). The infiltration trench purifies storm water prior to discharge into the Walker Dam Lake (see Appendix B.4 for more detail on site information and C.4 for detailed monitoring results).

### 3.3.5 Infiltration trench at housing estate Broxden, Perth

This end-of pipe system serves an extension of a housing estate located at Broxden, Perth. Road runoff is intercepted via typically trapped gully pots and then conveyed to the infiltration trench prior to discharge into the Craigie Burn. The two inspection chambers, which are built as sediment traps, are connected using two perforated pipes (see Appendix D.5). The inflow pipe distributes flow along the trench and is plugged at the downstream chamber. The outflow pipe is plugged at the inflow chamber and both plugs can be removed for access during maintenance (see Appendix B.5 for more detail on site information and C.5 for detailed monitoring results).

### 3.3.6 Infiltration trench at housing estate Transy Estate, Dunfermline

The surface water drainage at Transy Estate, Dunfermline serves roof and road surfaces of 16 houses. Runoff is intercepted via trapped gully pots and then conveyed to the end-of-pipe-infiltration trench, prior to discharge into the local sewer network.

The system incorporates two inspection chambers, one up and one downstream from the trench. The upstream chamber is built as sediment trap and incorporates a 225 mm diameter overflow-by-pass for extreme events. The system's outflow is regulated using a Hydrobrake. The inflow into the trench is distributed via three legs of 225 mm diameter perforated pipes (see Appendix D.6 for construction drawing, B.6 for more detail on site information and C.6 for detailed monitoring results).

## *3.4 Monitoring periods*

The monitoring programme started in January 2000 and ended in December 2003. Longer term monitoring was undertaken at the Lang Stracht system in Aberdeen for almost 3 years. The five additional systems were monitored for three to six months, each. Figure 3-2 is a bar chart of the monitoring period providing an overview of all monitored systems (for more detailed information see Appendix B).

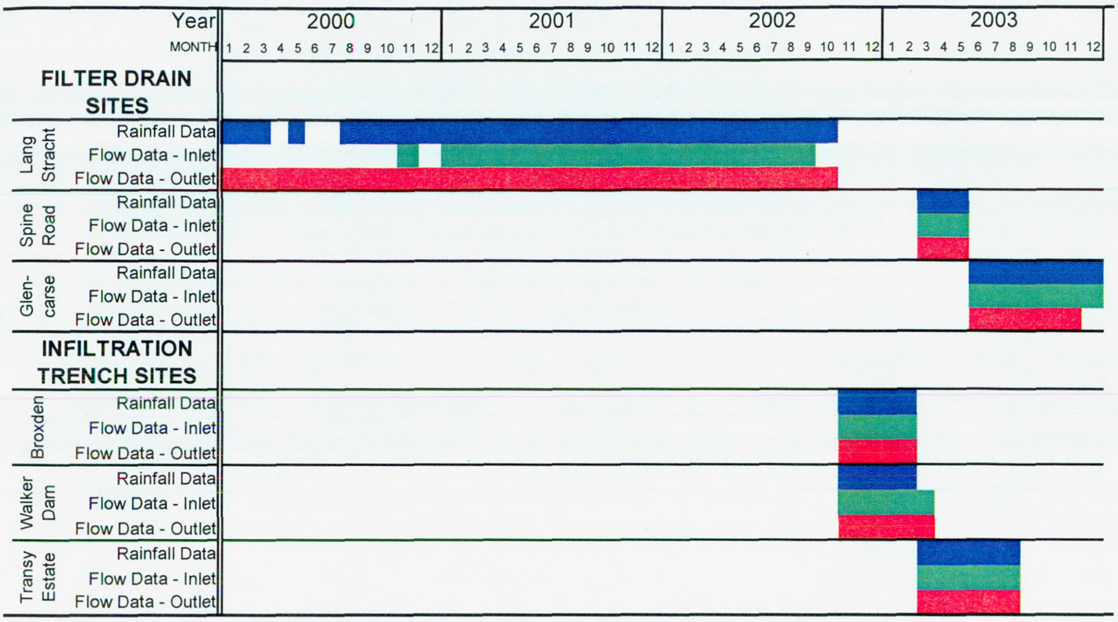



**Figure 3-2: Bar chart of the monitoring periods**

## *3.5 Instrumentation*

This section gives information on the usage of the monitoring equipment, including a description, and how it has been used for rain, runoff and quality data collection.

### 3.5.1 Rain data

*Tipping Bucket*

Tipping bucket rain gauges dates back to the 18th century and it is probably the most popular rain gauge today (Marsalek, 1981). The reason for such popularity comes from the very simple mechanism used for direct measurement of rainfall (La Barbera et al., 2002). Raingauges were installed away from any object that could impose shelter (i.e. distance to object more than 2 times its height). In addition, raingauges were not installed at locations exposed to wind. One raingauge was installed in close proximity to each of the monitored systems. Rainfall data was recorded as total number of tips in 2 minute intervals and site visits were undertaken once a month. Data was then downloaded and transferred to the Hydrol Time Series Manager database for further data analyses. Plate 3-1 shows the type of Casella rain gauge which was used. It is a reliable and robust transducer, designed as a stand-alone sensor with 0.2 mm tip sensitivity.


**Plate 3-1: Casella raingauge (Environmental Analytical Systems, 2002)**

### 3.5.2 Flow data

Three different instruments were used to observe a systems' hydraulic behaviour. These are a Sigma flow logger, a Vegason level monitor and a tipping bucket for surface runoff and these are described below.

*Sigma flow logger*

Two different types of Sigma Flow Loggers were used: The Sigma 950 and the Sigma 911. The Sigma flow loggers, as shown in Plate 3-2 (a) and (b), both use an ultrasonic Doppler sensor to measure velocity and a pressure transducer to measure level.

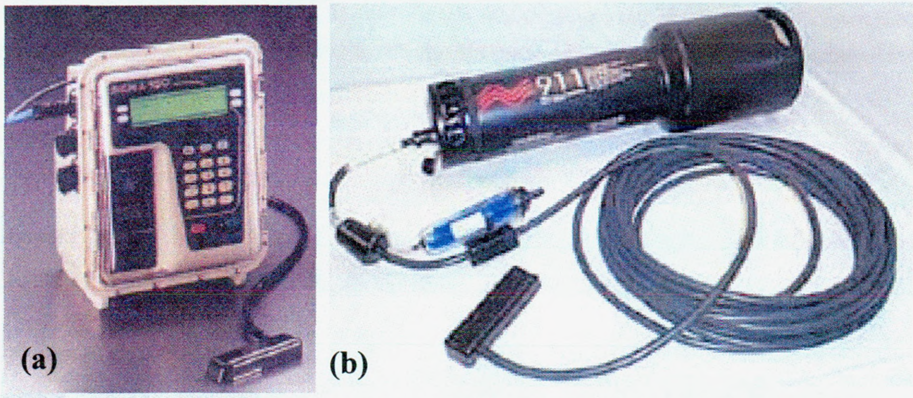



**Plate 3-2: Sigma 950(a) and Sigma 911(b) flow logger**

The ultrasonic sensor is installed at the bottom of the pipe. Flow velocity is measured with an ultrasonic Doppler signal continuously injected into the water. With this technique the instrument measures flow velocity with accuracy of ±2% (Bühler Montec, 2002).

The Sigma flow loggers were set up to record level and velocity every five minutes to allow for accurate data analysis. The velocity sensor was found to be unreliable when measuring low flows. Once the water levels dropped below 0.03m, the velocity sensor read zero. A head-discharge relationship was used to predict the flow from 0 to 0.03m of water level (see Section 3.6.4).

### *Vegason level monitor*

A Vegason 51K ultrasonic level monitor, in connection with an Isodaq VF1e single channel logger as shown in Plate 3-3, was used in addition to Sigma flow loggers to record level.

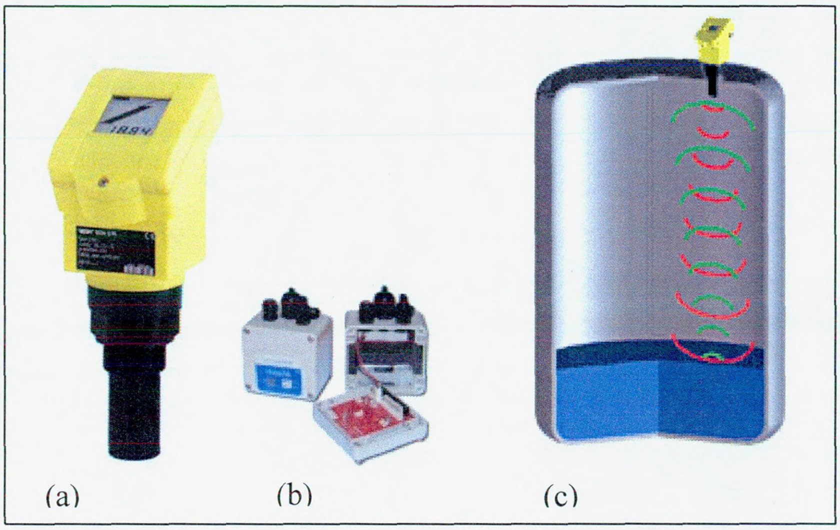



**Plate 3-3: Vegason 51K (a); Isodaq VF1e data logger (b); Schematic application (c)**

The Vegason 51K measures the height of solids or liquids in various environments and provides level measurement in the range from 250mm to 4m. High performance transducers emit ultrasonic pulses which are reflected from the measured surface. The Vegason calculates the level from the speed of the reflected pulses (Vega. 2002).

### *Tipping bucket for surface runoff*

This instrument was purpose built by the University of Abertay Dundee (UAD) to monitor extreme low flows, in the range from 0 to 1 $ls^{-1}$, which is typical for small road sections, where gully pots are used. Tipping buckets used to monitor surface runoff work on the same principle as a tipping bucket raingauge, the main difference being the size of the bucket, which measures 1 litre. Plate 3-4 (a) shows a picture of the tipping bucket manufactured at UAD and Plate 3-4 (b) a schematic of tipping bucket after Marsalek, (1981).

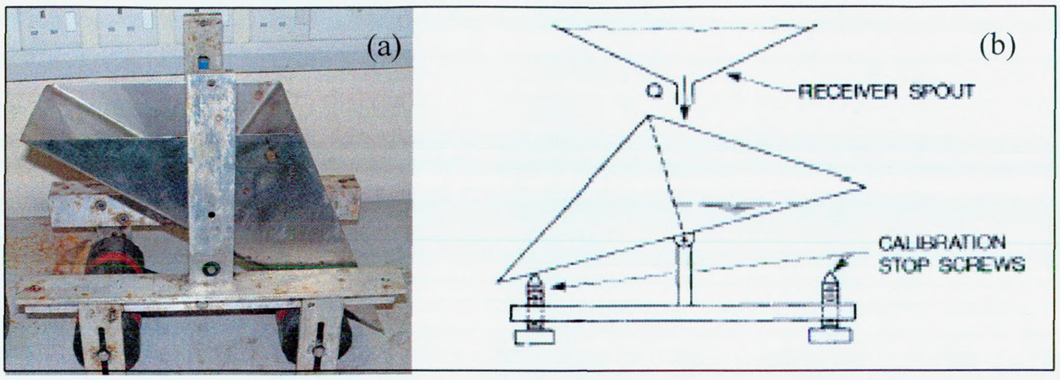



**Plate 3-4: Tipping bucket for surface runoff: (a) manufactured at UAD and (b) after Marsalek (1981)**

### 3.5.3 Water Quality data

Water quality monitoring was undertaken to support the assessment of the systems' performance. However, financial and time constrains resulted in the reduction of water quality monitoring and conclusive data is available from Lang Stracht, only. This added to conclusions drawn on the system's maintenance and monitoring results are presented in section 4.8.4 for Lang Stracht.

Two different methods were used to obtain water quality data

1. Solomat quality sonde
2. EPIC automated water sampler

#### *Solomat quality sonde*

Sondes were installed in the system's inlet as well as the outlet chamber. The primary purpose of this equipment was to give an indication of the change in the water quality. Data obtained from the quality sonde have to be interpreted as guidelines rather than exact values. Calibration checks and data comparison with water samples were undertaken to confirm correct data measurement. The sonde was set up to take spot readings of the following parameters every 5 minutes:

- Temperature
- pH
- Conductivity
- Dissolved oxygen
- Turbidity

Plate 3-5 shows the solomat sonde in operation.

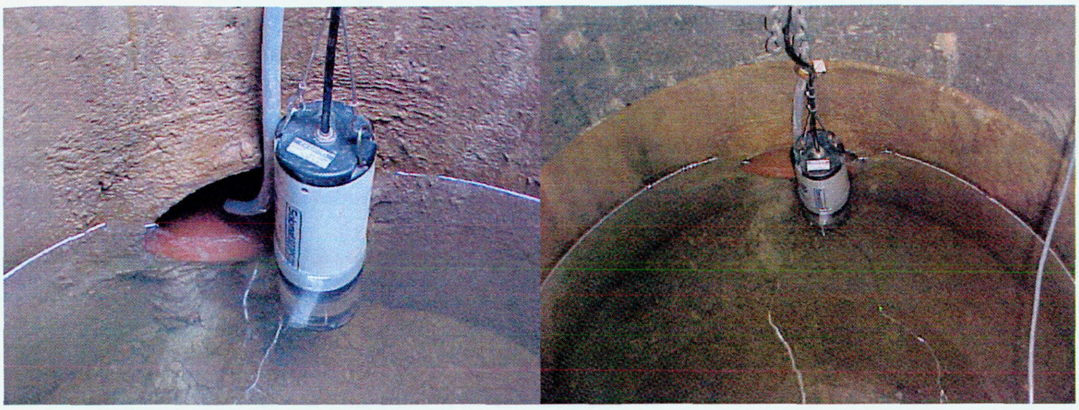


**Plate 3-5: Solomat sonde in operation**

### *Epic automatic water sampler*

The Epic automatic water sampler operates with 12 or 24 sampling bottles, depending on the sampling regime (see below). The sampler was set to trigger at a specified water level, recorded by the Vegason level monitor. Once started, the sampler took samples continuously at a preset time interval. Table 3-2 gives an example set-up of the Epic program.

| Shots per bottle | Time interval | Compensation sample per bottle | Number of bottles |
|---|---|---|---|
| 2 | 5 min | 10 min | 24 |

**Table 3-2: Typical set up for Epic sampler**

The Epic Sampler was set up to take water samples which were analysed for the following parameters:

- pH
- Conductivity
- Total suspended
- Solids
- Turbidity
- Chloride

## 3.6 Flow event analysis

Hydraulic results from on-site monitoring showed a flow volume reduction of 34-80% and peak flow reduction of 47-86%. Lag times varied between less than ½ hour at Broxden and Walker Dam to over 7 hours at Lang Stracht. Hydraulic analysis was undertaken at each monitoring site for commonly used parameters which characterise the hydraulic performance of the systems. This section provides a brief description of the hydraulic parameters, which are as follows:

- Event duration
- Lag time
- Antecedent Precipitation Index (API)
- Head-discharge relationship
- Percentage runoff
- Percentage flow volume
- Peak flow reduction
- Initial loss

### 3.6.1 Event duration

The duration of an event is determined by estimating an appropriate start and end point of time. The hydraulic characteristics of filter drains and infiltration trenches are such that small rain events rarely produce any outflow and there is no base flow. This was the main reason for undertaking the event analyses according to the systems' outflow. A flow event starts, once outflow occurs and ends when the outflow drops to zero. Corresponding rainfall data was selected manually and this may have occurred hours prior to the start of the outflow. Analysis was undertaken using an eyeball comparison of rainfall and flow hydrographs.

Once the event duration was estimated, events with a total rainfall depth of less than two millimetres were discarded from further analyses, as this rainfall depth typically corresponds to the initial runoff losses.

No fixed criteria were used to determine the event duration as every monitoring site is different and depending on the site characteristics and catchment wetness, rainfall may produce an instant system outflow or it may take hours until outflow occurs. It was therefore decide determine the event duration on a case-by case-basis (see Appendix C for a comparison of rainfall with outflow).

### 3.6.2 Lag time

Another measure to characterise the hydraulic performance of an in-ground SUD system is the lag time, which is calculated as the true difference between the centre gravity of the inflow hydrograph to centre gravity of the outflow hydrograph (Chadwick and Morfett, 1993). This calculation was carried out manually for each hyetograph and hydrograph by exporting data from the Hydrol Time Series Manager database to a spreadsheet using the following formula:

$$T_{CG} = \frac{\sum (A_i \bullet TD_i)}{\sum A_i} + T_{ES} \tag{3-1}$$

$T_{CG}$ is the time of centre gravity, $T_{ES}$ is the time of the start of the event, $A_i$ is the data value (rainfall intensity or flow) multiplied by the time-step and $TD_i$ is the duration from the event start up to each data point. This analysis was undertaken for all infiltration trench systems.

The filter drain systems receive inflow along their lengths and a lag time approximation is carried out which compares the centre gravity of the rainfall hyetograph with the flow hydrograph. This approximation is valid, as all filter drain systems are located immediately adjacent to their catchment areas and rain gauges were placed in close proximity to each system.

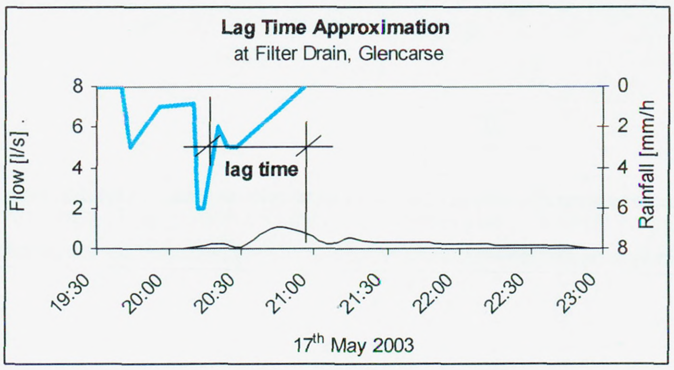



**Figure 3-3: Lag time approximation**

### 3.6.3 Antecedent precipitation index (API)

It is important to investigate the wetness of the catchment before an event in order to obtain correct data interpretation. A general guide to the degree of the catchment wetness can be obtained from the antecedent precipitation index after 5 days ($API_5$) (Shaw, 1989). The following equation defines $API_5$ at 9am on the day of the event.

$$API5_9 = \sum_{n=1.5} P_{-n} C_P^{n-0.5} \tag{3-2}$$

$P_{-n}$ is the depth of rainfall on day n before the event (measured 9am to 9am GMT in the UK) and $C_p$ is the decay coefficient, which equals to 0.5. Equation (3-3) gives $API_5$ at the time of the event.

$$API_5 = API5_9 C_P^{(t-9)/24} + P_{t'-9} C_p^{(t-9)/48} \tag{3-3}$$

t' is the time (hours) of the beginning of the event and $P_{t'-9}$ is the rainfall depth between time t' and 09:00 am.

### 3.6.4 Head-discharge relationship

Outflow from in-ground SUD systems is characterised by long periods of low flows, which are difficult to monitor with flow loggers which use the Doppler effect (see Section 3.5.2). During low flows, the measured velocity often drops to near zero although flow may still occur for a prolonged period of time and this may be of great significance for flow volume analysis. In order to improve the accuracy of data interpretation, a head-discharge relationship was developed at monitoring locations, which received flow from gravel fill material, resulting in long periods of low flows. An eye-fitted curve was used to predict flows once depth was so low that the velocity sensor read zero. The eye-fit was carried out to represent the head-discharge relationship. The eye-fit curve was chosen as the Hydrol Time Series Manager allowed the convenient application of this type of curve and also because this relationship did not follow mathematical relationships at every site, i.e. linear, exponential, log, etc. Figure 3-4 shows typical hydrographs of monitored flow in comparison to generated flow from a head-discharge relationship and Figure 3-5 shows a head-discharge relationship with an eye-fitted curve at Broxden.

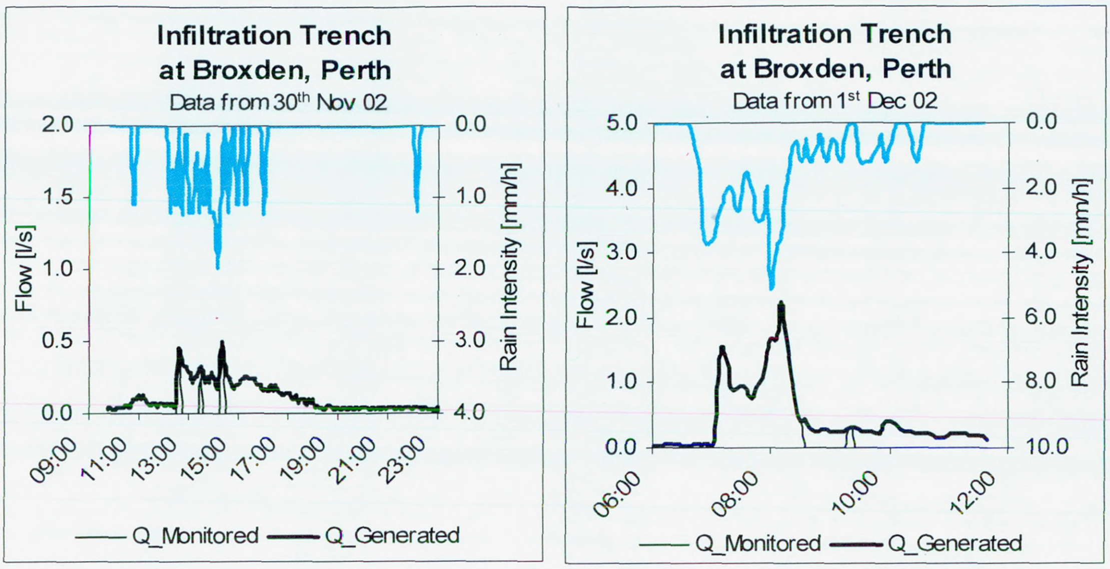



**Figure 3-4: Typical hydrograph of monitored and generated outflow**

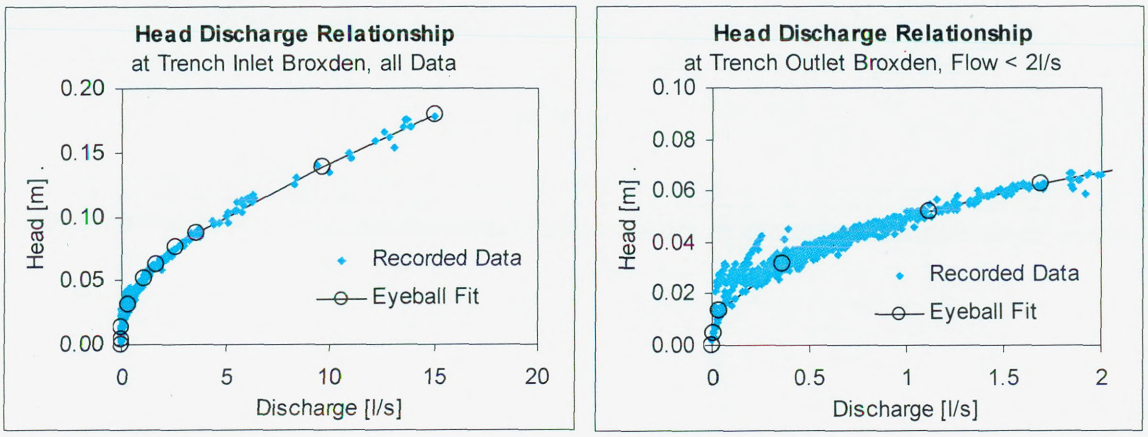



**Figure 3-5: Head-discharge relationship with eyeball fitted curves**

### 3.6.5 Percentage runoff

Percentage runoff is a commonly used parameter, which gives information on the volumetric discharge from the drained area and standard values range from 75% - 100% (BS8301) and this depends on the catchment type (i.e. roof or highway surface) and its characteristics.

Percentage runoff could be calculated from monitored results of infiltration trench systems but not from results of filter drains. The monitoring set-up at the filter drain systems did not allow for the calculation of this parameter, as inflow had already passed through filter media where flow volume loss occurred. The inflow to infiltration trench systems receives surface runoff from the drainage area, which allows the calculation of this parameter.

### 3.6.6 Percentage flow volume

The analyses for this parameter were carried out for both, infiltration trench and filter drain systems and this enabled a direct comparison between the different sites. However, the flow volume analysis for filter drain systems includes surface runoff losses, as the discharge volume is in comparison with rainfall volume. Whereas flow volume calculations from infiltration trenches describe the volumetric discharge in comparison with the systems' inflow, excluding any surface runoff losses.

### 3.6.7 Peak flow reduction

The peak flow reduction could be calculated for infiltration trench systems only. The system design of the filter drains did not allow monitoring of the inflow and no peak flow reductions could be calculated.

The peak flow reduction of the infiltration trench systems was calculated by comparing the peak of the outflow with the peak of the inflow per event.

### 3.6.8 Initial system loss

The sum of initial runoff losses, depression storage losses and losses within the in-ground SUD system is termed here 'initial system loss' and this is the amount of rainfall which is lost at the start of an event, prior to commencing system outflow. Comparing system outflow with rainfall this amount of rainfall loss can be estimated where the trendline crosses the y- axis (see Figure 3-6), (Schlüter and Jefferies, 2001). This analysis was undertaken for all monitored systems.

## *3.7 Monitoring results*

The performance of the investigated systems is presented here by comparing the monitoring results of each system (see Appendix C for more detailed information). Relationships between different parameters were found and analysis was undertaken according to section 3.6. All monitoring data, including hydrographs, event summary tables and attempted relationships are attached in Appendix C.

### 3.7.1 Overview of Monitoring Results

Table 3-3 provides a summary of monitoring results from all sites. The rain events at all locations had similar rainfall intensities and depths, which allowed for direct comparisons. The mean rainfall intensity ranged from 6 to 11 mm/h and total rainfall depth was from 6 to 10 mm. Monitoring results from Lang Stracht are shown for illustration purposes (see below).

The system at Broxden performed best in terms of percentage outflow and peak flow reduction and the system at Transy performed worst. The system at Transy was influenced by ground water ingress, resulting in frequent outflow of more than 100% (see Appendix B.6 and Appendix C.6).

The rainfall distribution at Spine Road and Walker Dam is thought to be the main reason for the outflow exceeds 100%.

| | Lang Stracht | | | | Spine Road | | | | Glencarse | | | | Broxden | | | | Walker Dam | | | | Transy | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Rainfall | | Outfow* | | Rainfall | | Outfow* | | Rainfall | | Outfow* | | Rainfall | | Outfow* | | Rainfall | | Outfow* | | Rainfall | | Outfow* | |
| | [mm/h] | [mm] | [l/s] | [%] | [mm/h] | [mm] | [l/s] | [%] | [mm/h] | [mm] | [l/s] | [%] | [mm/h] | [mm] | [l/s] | [%] | [mm/h] | [mm] | [l/s] | [%] | [mm/h] | [mm] | [l/s] | [%] |
| Min | 1 | 2 | 0 | 4 | 3 | 3 | 1 | 31 | 2 | 2 | 0 | 7 | 2 | 2 | 1 | 19 | 1 | 2 | 1 | 30 | 1 | 2 | 0 | 0 |
| Max | 12 | 62 | 11 | 81 | 30 | 38 | 18 | 110 | 72 | 13 | 21 | 95 | 30 | 15 | 4 | 41 | 42 | 19 | 9 | 123 | 60 | 21 | 6 | 264 |
| Mean | 7 | 10 | 2 | 29 | 11 | 10 | 4 | 63 | 11 | 6 | 3 | 47 | 6 | 6 | 2 | 27 | 10 | 8 | 4 | 66 | 11 | 6 | 1 | 70 |

* as peak flow in l/s and % of inflow volume

**Table 3-3: Summary of monitoring results**

The monitoring results from the system at Lang Stracht should not be compared directly with the other systems. Analyses of Lang Stracht data were only undertaken for events less than 13mm/h and remaining events had to be discarded (see Section 4.6.2 and Appendix B.1).

### 3.7.2 Flow volume

Flow volume is calculated in relation to catchment area and compared with total rainfall volume per event and relationships derived from monitoring results are shown in Figure 3-6. Graphs which show the monitored data and the origin of each relationship are attached in Appendix C.

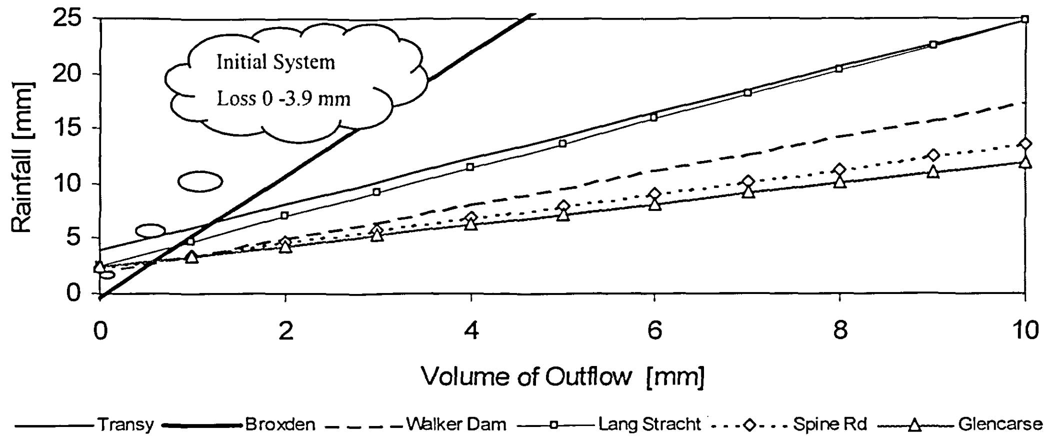



**Figure 3-6: Comparison of rainfall & flow volume**

The relationships from Figure 3-6 are presented in Table 3-4 and the raw data are provided in Appendix C.

| Site | Transy | Walker Dam | Glencarse | Spine Rd | Broxden | Lang Stracht |
|---|---|---|---|---|---|---|
| Formula | y = 2.1x + 3.9 | y = 1.6x + 2.0 | y = 0.95x + 2.5 | y = 1.1x + 2.4 | y = 5.5x - 0.3 | y = 2.2x + 2.6 |
| $R^2$ | 0.79 | 0.93 | 0.76 | 0.96 | 0.76 | 0.79 |

**Table 3-4: Formulae and $R^2$ relating rainfall & outflow**

The analysis for initial system loss (as described in Section 3.6.8) shows unexpected findings with Broxden providing zero loss and results from all other sites ranging between 2.0mm and 3.9mm (see Figure 3-6). The result from the system at Broxden is inconclusive and findings from the remaining sites are in line with data from a previous study by Schlüter and Jefferies (2001) and Macdonald (2003).

When using the formulae presented in Table 3-4 to compare the systems' performance of a rainfall event with 10mm rain depth, the infiltration trench systems at Transy, Broxden and Walker Dam reduced the outflow to 2.9, 1.8 and 5.17 mm, respectively. The filter drain systems at Lang Stracht, Spine Road and Glencarse produce outflow of 3.3, 6.1 and 7.9 mm for the same rainfall depth. The system at Broxden and Transy provide the highest flow volume reduction and the system at Glencarse and Spine Road were found to have the lowest flow volume reduction.

### 3.7.3 Peak flow

The peak flow discharge is most influenced by the rainfall depth and intensity. However, relationships were very weak at all systems and no overall pattern was found. This is thought to be due to the differences in rainfall characteristics.

Table 3-5 shows the peak flow reduction at monitored sites. This was calculated at infiltration trench systems only (see 3.6.7).

| | | Lang Stracht* | Spine Rd | Glencarse | Broxden | Walker Dam | Transy |
|---|---|---|---|---|---|---|---|
| Peak Flow Reduction | [%] | - | - | - | 77% | 47% | 61% |

**Table 3-5: Peak flow reduction**

Peak flow reduction was between 47 % and 77 %. The systems at Transy and Broxden performed better than the system at Walker Dam. The high peak flow reduction at Transy is unexpected as the system is influenced by groundwater ingress. However, monitoring results showed several events which did not produce any outflow and groundwater ingress was noted during prolonged wet periods only (see Appendix C.6). Comparing the system at Walker Dam with Broxden, indicates that the flow path may be the main reason for the improved performance. Although both systems have a similar catchment characteristics and storage volume, the system at Broxden is over 40 metres long with the system at Walker Dam is less than ¼ of that length.

### 3.7.4 $API_5$

The wetness prior to the start of the event was calculated as $API_5$ and this was found to have a significant influence on performance. A comparison of $API_5$ and total outflow over rainfall, which equals the flow volume in percent, is shown in Figure 3-7 with details of the functions in Table 3-6. The systems at Glencarse, Transy and Walker Dam produced realistic relationship and the systems' performances decreased with the increase of $API_5$.

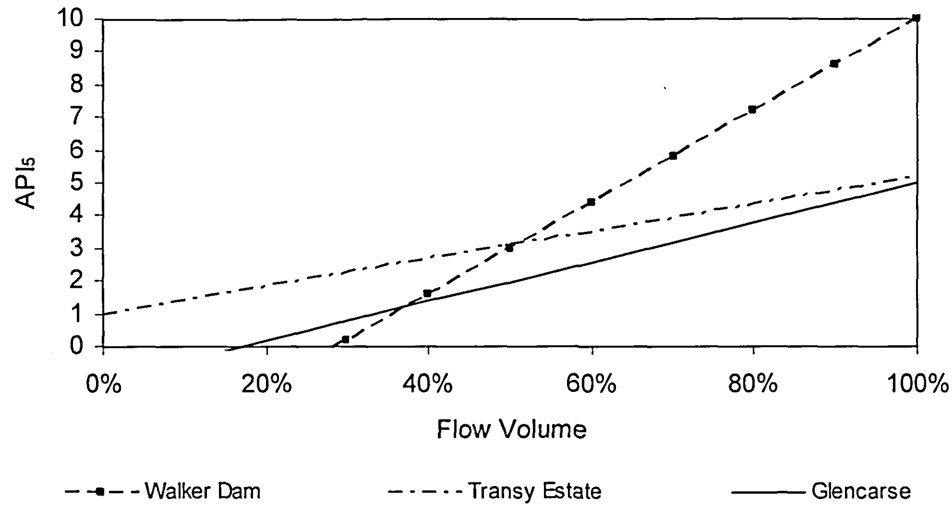



**Figure 3-7: Comparison of $API_5$ & flow volume**

The strongest relationship was found at Transy and this is thought to be due to the system's characteristics. The system at Transy receives flow from an additional trench and it is also influenced by ground water ingress during wet periods. These additional sources of inflow result in an increased influence of the system's wetness and this is the reason for the strong relationship with $R^2$ of 0.81 at Transy (see Appendix C6 for details of the monitoring results). Table 3-6 shows the relationships for Transy, Walker Dam and Glencarse.

| | Transy Estate | Walker Dam | Glencarse |
|---|---|---|---|
| Formula | $y = 4.2x + 1.0$ | $y = 14.0x - 4.0$ | $y = 6.0x - 1.0$ |
| $R^2$ | 0.81 | 0.44 | 0.75 |

**Table 3-6: Formulae and $R^2$ relating $API_5$ & percentage flow**

### 3.7.5 Lag time analysis

Section 3.6.2 explains the calculation of Lag time. Lag time is influence by the catchment wetness, rainfall depth and intensity. For roadside filter drain systems, the rainfall distribution may have an influence on the lag time, as inflow is along the system's length and dependent on the location of the data logger, lag times may vary considerably. Table 3-7 provides monitoring results from all systems. These values are averages for all monitored events and provide an indication of the flow attenuation in form of lag time for each monitoring site.

| | Lang Stracht | Spine Rd | Glencarse | Broxden | Walker Dam | Transy |
|---|---|---|---|---|---|---|
| Lag Time [hh:mm] | 07:20 | 01:22 | 00:48 | 00:28 | 00:27 | 03:00 |

**Table 3-7: Lag time**

It can be seen that the lag time at the filter drain systems was much longer than the infiltration trench systems. Lang Stracht has the longest lag time and Walker Dam and Broxden the shortest. The increased lag time at the infiltration trench system at Transy is thought to be due to the influence of ground water ingress and this is shown for illustration purposes only.

The approximation of the lag time at filter drains systems (see Section 3.6.2) may also have an influence on this analysis, (i.e. Lag time is calculated dependent on the rainfall rather than inflow). Dependent on the proximity of the raingauge to the catchment results may vary slightly. In addition, the flow time of system entry could be accounted for two to five minutes of increased lag time and this depends on the catchment characteristics such as dimension, slope and rainfall losses. The system dimension, pipe arrangements, slope, type of fill material, soil type and system wetness will then influence the time of flow travelling through the system.

The catchment wetness, calculated as $API_5$ was found to have an influence on the lag time and a relationship was found at three sites. Although relationships were relatively weak, a distinctive pattern was found and Figure 3-8 and Table 3-8 shows the results.

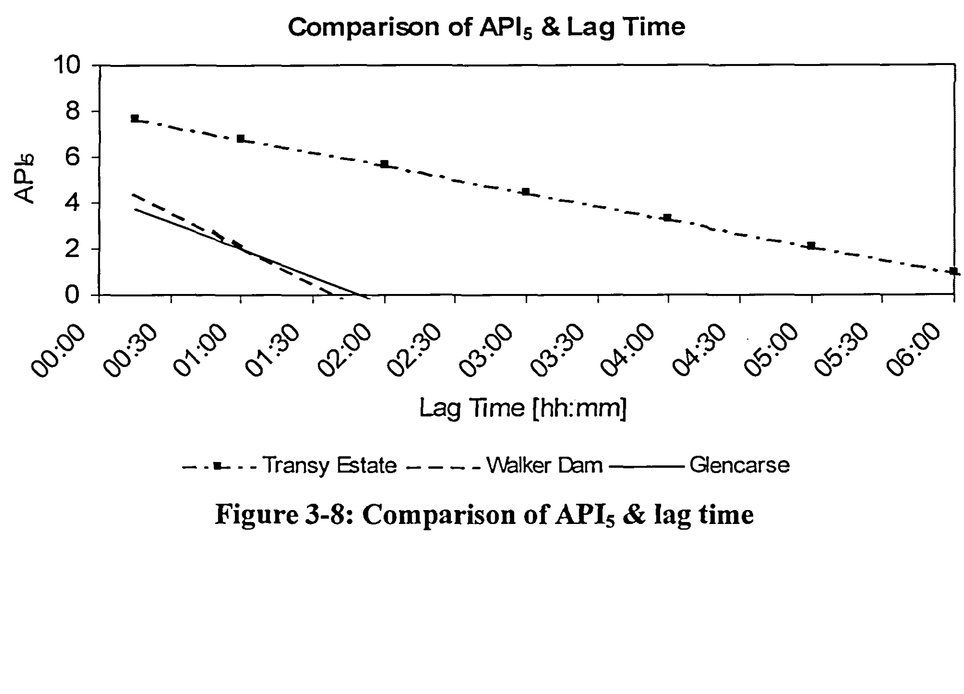



**Figure 3-8: Comparison of $API_5$ & lag time**

| | Transy Estate | Walker Dam | Glencarse |
|---|---|---|---|
| Formula | y = -27.9x + 7.9 | y = -74.6x + 5.2 | y = -57.4x + 4.4 |
| $R^2$ | 0.53 | 0.38 | 0.21 |

**Table 3-8: Formulae and $R^2$ relating $API_5$ & lag time**

The strongest relationship was found at Transy and weakest relationship at Glencarse. The influence of ground water ingress at Transy is thought to be the reason for a stronger influence of the catchment's wetness.

## *3.8 Discussion*

Results from hydraulic monitoring of typical in-ground SUDS provided detailed information of the performance of each system. Hydraulic analysis was undertaken at each monitoring site for commonly used parameters which characterise the hydraulic performance of the systems. Flow volume reduction was calculated by comparing the system inflow and outflow at infiltration trench systems. The system design of filter drains did not allow inflow monitoring. However, flow volume reduction of filter drain systems was approximated by comparing rainfall depth with outflow. Results showed that system behaviour of the filter drains was similar to the performance of the infiltration trenches (see Figure 3-6). Flow volume reduction was found to be between 34-80% and peak flow reduction 47-86%. Lag times varied between less than ½ hour at Broxden and Walker Dam to over 7 hours at Lang Stracht. All systems, apart from Broxden, provide an initial system loss between 2.0 and 3.9 mm and these are in line with findings from a previous study by Schlüter and Jefferies (2001) and Macdonald (2003).

The system at Broxden performed best in terms of flow volume and peak flow reduction and this is thought to be due to good drainage soil and the excellent system design (see Appendix D 4). No information of the infiltration rate was available for any of the systems. However, hydraulic modelling was used to estimate the infiltration rate and this is detailed in Chapter 5.

The system at Transy shows features of good flow control and good system design as it makes use of a vortex flow control at the downstream end and flow is well distributed using three perforated pipes (see Section 4.5.3). However, flow monitoring at this site showed unexpected findings as the system was influenced by groundwater ingress during prolonged wet period (see Appendix B.6 and F.6) and this was the reason for outflow in excess of 100%. These findings show the need for a thorough inspection of the ground conditions including measurement of seasonal ground water levels. Infiltration trench systems are unsuitable at locations with high ground water levels (see Section 2.4.3).

Long-term monitoring at Lang Stracht revealed hydraulic failure due to frequently over spilling gully pots resulting in runoff not entering the filter drain but causing temporary flooding on the road. Events with more than 12mm/h rainfall intensity were expected to cause the gully pots to overspill and were therefore excluded from further investigation (see Section 4.6.2 for more information).

Only weak relationships between rainfall, outflow, lag time and catchment wetness ($API_5$) were found. This is thought to be due to the complexity of the system, where various parameters influence the system's performance.

The change in water level within the filter material was recorded at Walker Dam, only. The design of the remaining systems did not allow monitoring of the water level within the filter material. Results of this measurement at Walker Dam showed that an infiltration trench system can operate satisfactory despite being located in low permeable soils with prolonged emptying times (see Section 5.9 for more information).

Most systems were located in soils of low permeability but no detailed information about the infiltration rates was available. To better understand the system's hydraulic behaviour and to undertake a performance comparison when using design rainfall, generic models were needed. Model development of generic models and a performance comparison between the different systems is shown in Chapter 5.

Water quality monitoring was undertaken at four systems but no results are presented here. Monitoring results from the system at Lang Stracht was used in Section 4.8.4 to illustrate the impact of maintenance.

The experience gained from on-site monitoring provided detailed characteristics of the systems' behaviour which enabled a performance assessment of additional sites. Additional sites were assessed during site visits, manhole entry and CCTV surveys and this is shown in the following section.

# CHAPTER 4 VISUAL ASSESSMENT OF IN-SITU SYSTEMS

*Information gathered during the survey of more than 40 filter drains and infiltration trenches is assessed in this section. An overview of the systems under investigation is given and findings from the survey are evaluated using a numerical scoring system. The information has been categorised and examples of good and bad practice drawn out. A discussion on detailing is given at the end of this section. Results from this assessment are used later to provide recommendations for improved detailing and maintenance of in-ground SUDS.*

## 4.1 Rationale

The objective of this part of the research was to inspect as many sites as possible to gain a wide overview of in-ground SUDS installations in Scotland and to assess their general performance. This information was used to identify detailing of good and bad design. In addition, maintenance procedures and intervals are proposed for most systems and this provides vital information for authority and highway operators, which are responsible for a system's long-term performance and maintenance. To enable a numerical comparison between the systems and to aid identifying systems of good practice, a scoring system is introduced.

## 4.2 Methodology

Appropriate developer, water authority and SEPA personnel were contacted for information on system concepts, record plans and for other information.

In addition to flow monitoring of six systems (see Chapter 3), another 37 sites were inspected by manhole entry and general visual inspection. Fifteen sites were selected for CCTV inspection from within the perforated drainpipe.

Monitoring results, findings from the visual inspection in combination with record drawings etc. and results from the CCTV survey were evaluated and a rating system was introduced to enable inter-system comparison and to emphasise each system's good and bad design.

Maintenance procedures from highway operators and Water Authorities were assessed. These organisations undertook routine inspections and responded to emergent problems with a preventative maintenance programme. Otherwise, the maintenance programme and frequencies proposed were based on informed estimates of the likely accumulation of sediment and contaminants.

The scoring system enabled a comparison between the sites and scores were given for various criteria relating to water quality and flow attenuation, detailing and maintenance. The overall score provides an indication of the system's performance.

## *4.3 Overview of systems under investigation*

A total number of 43 sites were investigated, the majority being systems located in small to medium size housing developments.

The average age of the sites examined was 4 years, the oldest and youngest being 10 and 1 years, respectively. One third were more than five years old and two thirds less than five years old. The distribution of all investigated systems is given in Figure 4-1 and an overview of information is given in Table 4-1 with a legend in Table 4-2. For more detailed information see Appendix E. The systems are sorted alphabetically according to their location and references to their System No are given during the following section of good and poor practice.

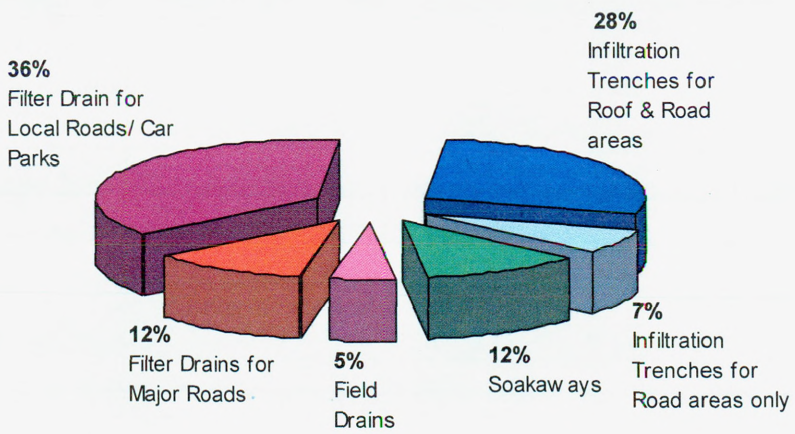



**Figure 4-1: Distribution of systems investigated**

| Number [-] | Locatin [-] | Site [-] | Overall Rating [-] | Catchment Area [m$^2$] | Catchment Type* [-] | Houses connected [-] | Type of SUD System** [-] | Type of Inlet*** [-] | Treatment Volume [m$^3$] | Overflow [-] | Receiving Water [-] |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Aberdeen | Lang Stracht | 1 | 9520 | MR | 0 | FD | TGP | 137 | Yes | Sewer System |
| 2 | Aberdeen | Woodend | 1 | 13000 | H,LR | 42 | EP IT | TGP | X | No | Burn of Rubislaw |
| 3 | Aberdeen | Arnhall Business Park | X | X | LR | 0 | FD | TGP | X | X | Burn |
| 4 | Aberdeen | Elrick Road | X | X | LR | 0 | FD | TGP | X | X | Burn |
| 5 | Aberdeen | Arnhall Business Park | X | X | CP | X | FD | OK | X | X | X |
| 6 | Aberdeen | Great Northern Rd | X | X | F | X | FD | LF | X | X | X |
| 7 | Aberdeen | Walkerdam | 4 | 3424.5 | H,LR | 15 | EP IT | TGP | 7 | No | Walker Dam (Lake) |
| 8 | Blairhall | Housing Estate | 4 | 1285 | H,LR | 6 | EP IT | TGP | 24 | Yes | Burn |
| 9 | Dundee | Swallow Roundabout | 1 | X | MR | 0 | FD | TGP | X | No | Burn |
| 10 | Dunfermline | Spine Rd, DEX | 3 | 3057 | LR | 0 | FD | TGP | 155 | No | LyneBurn |
| 11 | Dunfermline | Transy Estate | 4 | 6568 | H,LR | 15 | EP IT | TGP | 127 | Yes | Sewer |
| 12 | Dunfermline | Queens Gate | 1 | 200000 | H,LR | 158 | EP IT | TGP | 69 | Yes | Sewer System |
| 13 | Dunfermline | Woodmill Road | 3 | 460 | LR | 0 | FD | TGP | X | No | Sewer System |
| 14 | Dunfermline | Tesco Car park, DEX | 1 | X | F | 0 | FD | LF | X | No | Burn |
| 15 | Dunfermline | Spine Rd, DEX | 3 | X | LR | 0 | FD | OK | 49 | No | LyneBurn |
| 16 | Dunfermline | Pitreavie | X | X | LR | X | FD | TGP | X | X | X |
| 17 | Edinburgh | Forth road bridge North | 3 | X | MR | 0 | FD | LF | X | Yes | Burn |
| 18 | Edinburgh | Forth road bridge South | 3 | X | MR | 0 | FD | LF | X | Yes | Burn |
| 19 | Falkirk | Retail Park at Queens St | X | 21000 | CP | 0 | FD | TGP | X | Yes | Sewer System |
| 20 | Falkirk | Wallacelea Rumford | 2 | 3400 | LR | 0 | EP IT | TGP | 35 | Yes | Burn |
| 21 | Falkirk | Maddiston California Rd | X | 5651 | H,LR | X | FD | TGP | 72 | No | Burn |
| 22 | Falkirk | Wagon Road, Polmont | X | 7120 | H,LR | 180 | EP IT | TGP | X | Yes | Burn |
| 23 | Falkirk | BusinessPark, Larbert | 3 | 2280 | CP | 0 | FD | LF | 15 | No | Sewer System |
| 24 | Falkirk | Braes Highschool, Polmont | X | 5000 | H,LR | 0 | FD | TGP | 46 | Yes | Burn |
| 25 | Falkirk | Queens Drive Larbert | X | X | CP, LR | 0 | FD | TGP | X | No | Sewer System |
| 26 | Falkirk | Glenbo, Dennyloanhead | X | X | H,LR | 37 | EP IT | X | X | No | Sewer System |
| 27 | Falkirk | Taymouth Rd, Heatherlea | 3 | 935 | LR | 0 | FD | TGP | 1 | Yes | Sewer System |
| 28 | Findochty | Netherton Farm | X | 675 | H,LR | X | EP IT | TGP | 19 | Yes | Sewer System |
| 29 | Hatton | Lairds Park, Fintray | 4 | 7400 | H,LR | 23 | S, FD | TGP | 183 | Yes | Soil Infiltration |
| 30 | Invergowrie | Invergowrie Mill | X | X | H,LR | X | EP IT, S | TGP | X | X | Burn |
| 31 | Kirkhill | Albyn | 2 | 9644 | H,LR | 25 | S, FD | TGP | 60 | No | Burn |
| 32 | Perth | Broxden | 5 | 3300 | H,LR | 14 | EP IT | TGP | 16 | Yes | Craigie Burn |
| 33 | Perth | Glencarse | X | 7061.5 | MR | 0 | FD | LF | 149 | No | Burn |
| 34 | Perth | St Madoes Old | 3 | 2129 | LR | 0 | EP IT | TGP | X | No | Cairnie Pow |
| 35 | Perth | St Madoes New | X | X | H,LR | X | FD | TGP | X | No | Burn |
| 36 | Perth | Faries Rd | X | 3872 | H,LR | Varies | FD | TGP | X | X | X |
| 37 | Tayport | Sandy Hill | 2 | 392 | LR | 0 | FD | TGP | 25 | Yes | Burn |
| 38 | Tayport | Sandy Hill, S1 | 3 | 2205 | H,LR | 20 | S | TGP | X | Yes | Burn |
| 39 | Tayport | Sandy Hill, S8 | 3 | 2016 | H,LR | 21 | S | TGP | X | Yes | Burn |
| 40 | Tillicoultry | Tillyflats Grangemouth | X | X | LR | 0 | FD | X | X | X | Burn |
| 41 | Tullicoultry | Fir Park | 1 | 3610 | LR | 0 | EP IT | TGP | 27 | Yes | Burn |
| 42 | Westhill | Leddach Farm New | X | 2200 | H,LR | 9 | EP IT | TGP | X | Yes | Burn |
| 43 | Westhill | Leddach Farm Old | 2 | X | H,LR | X | EP IT | TGP | X | X | Burn |

**Table 4-1: List of all systems**

| Type of Catchment | | | | | | Type of SUD System | | | | Type of Inlet | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| * | MR<br>Major Road | LR<br>Local Road | CP<br>Car Park | H<br>House Area | F<br>Field | ** | EP IT<br>End-of-pipe Infiltration Trench | RS FD<br>Road side Filter Drain | S<br>Soakaway | *** | TGP<br>Typical Trapped Gully Pot | OK<br>Offlet Kerb | LF<br>Lateral Flow |

**Table 4-2: Legend for Table 4-1**

This survey of in-ground SUDS showed that almost 75% of all systems discharge to natural watercourses, disconnecting a significant amount of impermeable area from combined sewer systems. Catchment areas varied from 392m$^2$ to 200,000m$^2$, typically consisting of road and roof surfaces from small to medium size housing developments in addition to major roads. High-level by-passes or overflows are used to ensure hydraulic performance in case of extreme rainfall events. Overflows were found at more than 50% of all systems. There is a growing concern in the Water Authority and SEPA that many systems are permanently blocked, which may never be noticed at locations with overflows. The survey showed that more than 30% had signs of temporary blockage and one site was found which was permanently blocked.

Runoff from unstabilised areas or construction site runoff was found to be affecting the longevity of in-ground SUDS. Almost 30% of all sites were affected by construction runoff. Construction runoff at one location had blocked the inlet to the trench completely and all flow by-passed the filter.

The treatment volume as shown in Table 4-1 was approximated to be the storage volume, calculated as the volume of the trench with a gravel fill material of 30% porosity and neglecting any additional volume from manholes or pipes. A relationship between the catchment area and the system's treatment volume was found and this is presented in Figure 4-2.

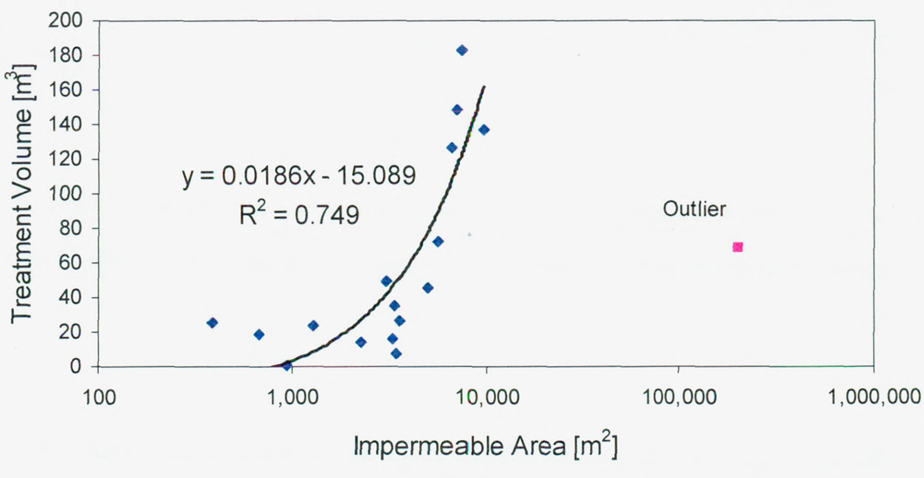



**Figure 4-2: Relationship between catchment area & treatment volume**

System No 12 was excluded from this calculation, shown as an outlier in Figure 4-2, as it was designed to treat the first flush, only.

## 4.4 The Schlüter scoring system for in-ground SUDS

A numerical procedure was developed to aid identifying systems of good and poor practice and to indicate the hydraulic and water quality performance. This procedure, which is named the Schlüter scoring system, uses specifically selected criteria relating to the system's water quality, hydraulic performance, detailed design and maintainability. Figure 4-3 shows the procedure which was used to obtain the score for each site and Table 4-4 to 4-8 show the details of the criteria.

The definition of system failure may depend on various characteristics and can be very site specific. However, for the purpose of this study and to enable an overall comparison between the various system failures it is described as when a system does not provide flow attenuation or improve water quality. The option to score the system's performance as failure was included for all criteria. If the system failed (i.e. scored 1) in criteria 1 or 2, the system failed overall and this is due to the severity of these two criteria, independent of the performance of the remaining ones. Although the option to fail was also included for criteria 3 to 5 individually, generally the system did not score an overall failure in these categories. Unless, the system failed in criterion 1 or criterion 2, the score was calculated to be the arithmetic average of the five scores. Detailed results from the scoring system are provided in Appendix E.2. Table 4-3 shows the scores and the categories associated.

| Score | 4.5 - 5.0 | 3.5 - 4.4 | 2.5 - 3.4 | 1.5 - 2.4 | 1.0 - 1.4 |
|---|---|---|---|---|---|
| Category | Excellent | Good | Poor | Fair | Failure |

**Table 4-3: Details of scores and categories**

Criterion 1 aims to evaluate a system's hydraulic performance by focusing on its ability to receive, distribute and attenuate flow. Knowledge gained from the hydrological investigation was taken into account and the main emphasis was given to findings from visual inspections. Table 4-4 shows parameters for criterion 1.

| Criterion 1 | Score |
|---|---|
| Near zero inflow capacity, i.e. almost all water by-passes or surcharges the system. No flow control/ attenuation. | 1 |
| Limited inflow capacity, i.e. frequent system surcharge/ by-passing system. | 3 |
| Good flow inflow distribution. Maximised inlet pipe length. Downstream flow control for flow attenuation. | 5 |

**Table 4-4: Factors relating to the hydraulic performance; C1**

Criterion 2 aims to evaluate a system's ability to retain pollutants and provide water quality improvements to the receiving water. Again, findings from the visual inspection including CCTV investigations were the main source of information in addition to experience from the limited water quality measurements and results from the onsite clean-out tasks. Table 4-5 shows the parameters considered for criterion 2.

| Criterion 2 | Score |
|---|---|
| Sediments/ pollutants visible in outlet chamber (sediment breakthrough). Polluted flow discharging to receiving water. | 1 |
| Significant accumulation of debris in inlet chamber and inlet pipe but no sediment breakthrough at outlet. | 3 |
| Clean flow pipes, very little sediment accumulation in inlet chamber, no pollutant accumulation in outlet chamber | 5 |

**Table 4-5: Factors relating to water quality performance; C2**

The detailing in relation to the system's ability to improve the water quality and attenuate flows was evaluated respectively, with criteria 3 and 4. The main sources of information were construction drawings, hydrological monitoring and visual inspections. Table 4-6 and Table 4-7 show details of the criteria 3 and 4.

| Criterion 3 | Score |
|---|---|
| No inlet sump, inflow pipe is connected to outlet, drop prior to flow entering the system. Sediments/ pollutants are flushed through the system. | 1 |
| Small sump provided, one perforated pipe, raised system outlet. System provides some pollutant retention. | 3 |
| Sedimentation sump at inlet, high level outlet, dip plate, inflow pipe and outflow pipe are disconnected. System provides excellent water quality features. | 5 |

**Table 4-6: Factors relating to water quality; C3**

| Criterion 4 | Score |
|---|---|
| Limited inflow capacity causing frequent surcharge/ flooding, i.e. short inlet pipe. | 1 |
| Sufficient inflow capacity but signs of debris accumulation that may reduce inflow. | 3 |
| Excellent flow distribution with downstream throttle to maximise storage volume, i.e. fishbone system with downstream hydrobrake. | 5 |

**Table 4-7: Factors relating to flow control; C4**

Criterion 5 evaluates the maintainability of the system. This was undertaken by assessing the maintenance procedures carried out in relation to results from the hydrological monitoring. Construction drawings, visual inspection and assessment of the maintenance procedures were the main sources of information. Table 4-8 shows details of criterion 5.

| Criterion 5 | Score |
|---|---|
| Not maintainable, no access chambers or rodding eyes. | 1 |
| Manhole at one end of the system, which limits the ability to flush through pollutants. | 3 |
| Manhole at both ends of the system and access to both, the inlet and outlet pipe. | 5 |

**Table 4-8: Criterion of maintainability; C5**

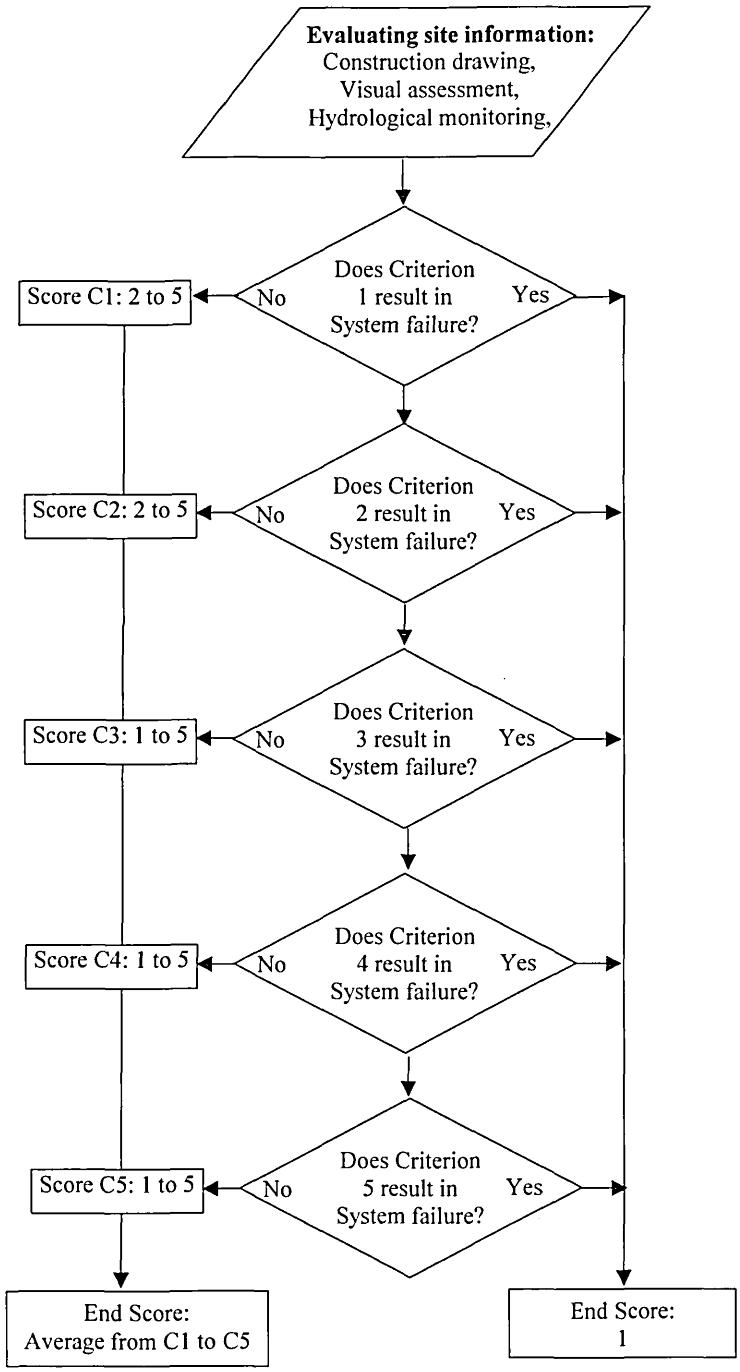



**Figure 4-3: Flow chart of scoring system**

26 of the 43 sites were ranked according to Table 4-3 and Figure 4-3. No score was given for the remaining 17 sites, as there was too little information. Results are presented in Figure 4-4. Table 4-9 shows the data from Figure 4-4 and also gives an example description for each score.

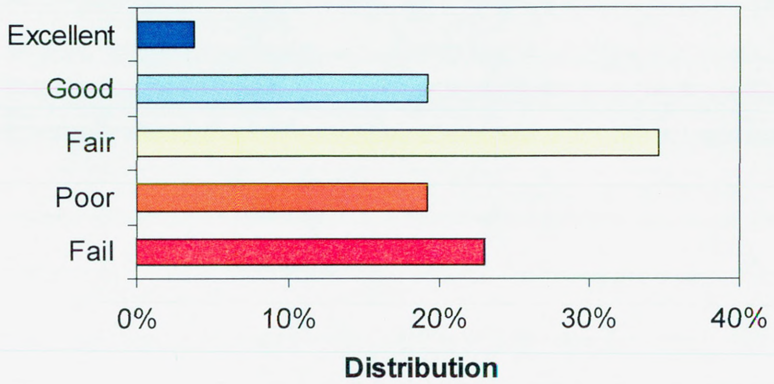



**Figure 4-4: Results from overall scoring**

| Score | Category | Results | Example Description |
|---|---|---|---|
| 4.5 - 5.0 | Excellent | 4% | System with excellent detailing, promoting flow attenuation and pollutant retention. System is well maintainable. |
| 3.5 - 4.4 | Good | 19% | System provides good flow attenuation and pollutant retention. Some maintenance required. |
| 2.5 - 3.4 | Fair | 35% | Does provide some pollutant retention and little flow attenuation. May show significant sedimentation and may need maintenance |
| 1.5 - 2.4 | Poor | 19% | Poorly designed with expectation to fail. Maintenance urgently required |
| 1.0 - 1.4 | Failure | 23% | Blocked system or system with sediment breakthrough at outlet, i.e. pollutants travelling through the system |

**Table 4-9: Scoring system for performance comparison**

Results show that 35% of the systems provided fair performance and in 19% the performance was good. Only one system was found to be performing excellently. A high failure rate of 23% was discovered and 19% were rated as poor (see Section 4.5 and 4.6 for examples of good and poor practice). Findings from this assessment with almost ¼ of all systems rate as Failure and an additional 19% rated as Poor give an indication of the poor performance of in-ground SUDS in Scotland and clearly show the need for improved detailing and maintenance procedures. Results also give an indication of the maintenance requirements and this is assessed in more detail in section 4.8.

The scoring system would have produced slightly different results if the overall score had been the average of all criteria without the option to fail from just one criterion. The number of systems with Fair and Poor performance would increase from 54 to 58% in total and the number of failed systems decrease from 23 to 4%. The number of systems with a good and excellent score would remain the same. However, it is thought that the scoring system with the option to fail using criteria 1 and 2 provides a better representation of the actual performance, otherwise blocked system or systems with highly turbid outflow would get a poor or even a fair performance score, which is thought to be unrealistic, i.e. a system which receives no flow due to blocked inlets can not provide poor or even fair performance.

Different, and in this case slightly better, results would have been produced if the scoring procedure was undertaken only for criteria 1 and 2. The percentage of systems performing excellently would increase from 4 to 15 % and the percentage of systems which previously failed would decrease from 23 to 19%. The total percentage of fair and poor performance remains at 58% and the percentage of systems which achieved a good score decreases from 19 to 12%. It could be argued that this scenario provides a better indication of the current situation of in-ground SUDS as it is based purely on the visual site assessment without taking account of the detailed design and maintainability (criteria 3 to 5). However, many systems were fairly new and although they may have appeared to be clean and in good working order, the maintainability and detailing would have had a significant influence on their future performance and this should also be taken into account (see criteria 3 to 5).

There were several difficulties associated with the scoring system. One main difficulty was the variety and type of information available and how to measure the information to provide scores for each criterion. Some sites were monitored in detail for long periods of time while other sites were assessed on the basis of only one site visit. Construction or design drawings were available for most of the system but some of them were also inspected by CCTV inspection. For 17 sites information was too sparse and these were excluded from the scoring system. The remaining 26 sites were assessed by considering all of the information available. Providing scores for criterion 1 and 2 were more difficult than for criteria 3, 4 and 5. Criteria 3 to 5 relate to the detailed design of each system and given that the system was built according to the design drawing, this was the main source of information, which was used for the scoring system (see Section 4.5 and 4.6 for examples of good and poor detailing). Scores for criteria 1 and 2 were based on the knowledge gained from the on-site monitoring which gave confidence on providing a judgement for systems which were assessed from visual inspections, only.

There are two main reasons why results from the scoring system have to be interpreted with caution. Firstly, results are based on less than 10% of the total number of installations in Scotland and, secondly, monitoring and in-depth investigation was undertaken for less than ¼ of the investigated systems. However, proposed performance results provide an indication of the effectiveness of installation and give an idea about the good and bad detailing from the investigated systems. Examples of good and bad detailing are presented in Chapter 4.5 and 4.6.

In order to provide more confidence in results from the proposed Schüter scoring system a conventional Risk Assessment was carried out and this is presented in Section 4.4.1

### 4.4.1 Environmental Risk Assessment

A semi-qualitative risk assessment was carried out to identify the hazards and environmental risks which are associated with each system and to enable a comparison of the proposed Schlüter Scoring methodology. The risk assessment was undertaken for the two most significant categories for in-ground SUDS, firstly, the risk of flooding and, secondly, the risk of water quality pollution of the downstream watercourse. Additional categories, such as ground water pollution or the amount of ground water recharge are also of importance but too little information was available. The procedure followed the description in Section 2.6 and details of the two categories are presented in Table 4-10. These were categorised by Probability according to Table 4-11.

| | Severity of Flooding |
|---|---|
| 1 | Property flooding |
| 2 | Road flooding |
| 3 | Local greenfield flooding |
| 4 | Bypass operating |
| 5 | System surcharging |

**(a)**

| | Severity of Water Quality Pollution |
|---|---|
| 1 | High turbidity outflow. Heavy metals & hydrocarbins expected. |
| 2 | High turbidity outflow. No heavy metals & hydrocarbins expected. |
| 3 | Increased turbidity outflow. |
| 4 | Slight turidity outflow. |
| 5 | Clean outflow. |

**(b)**

**Table 4-10: Severity categories of (a) Flooding and (b) Water Quality Pollution**

| | Probability |
|---|---|
| 1 | Once per 6 Months |
| 2 | Once per Year |
| 3 | Once per 2 Years |
| 4 | Once per 5 Years |
| 5 | Once per 10 Years |

**Table 4-11: Probability of occurrence**

Both Severity and Probability were plotted on a priority chart as shown in Table 4-12 to enable the numerical risk assessment.

| | | | Severity | | | | |
|---|---|---|---|---|---|---|---|
| | | | Very Severe | | Low Consequences | | |
| | | | 1 | 2 | 3 | 4 | 5 |
| Probability | High | 1 | 1 | 2 | 3 | 5 | 7 |
| | | 2 | 2 | 4 | 6 | 7 | 8 |
| | | 3 | 3 | 5 | 7 | 8 | 9 |
| | | 4 | 4 | 6 | 8 | 9 | 10 |
| | Low | 5 | 5 | 7 | 9 | 10 | 10 |

**Table 4-12: Priority matrix of the semi qualitative risk assessment (Pritchard, 2000)**

The numbers inside the boxes (Table 4-12) represent priority ranking and influence what risk control actions should be taken (e.g. Severity 3 & Probability 3 would be Priority 7). Action should normally be taken to mitigate the probability or reduce the consequences of hazards having priority 7 or lower (Pritchard, 2000). Results from this risk assessment are presented in Figure 4-5.

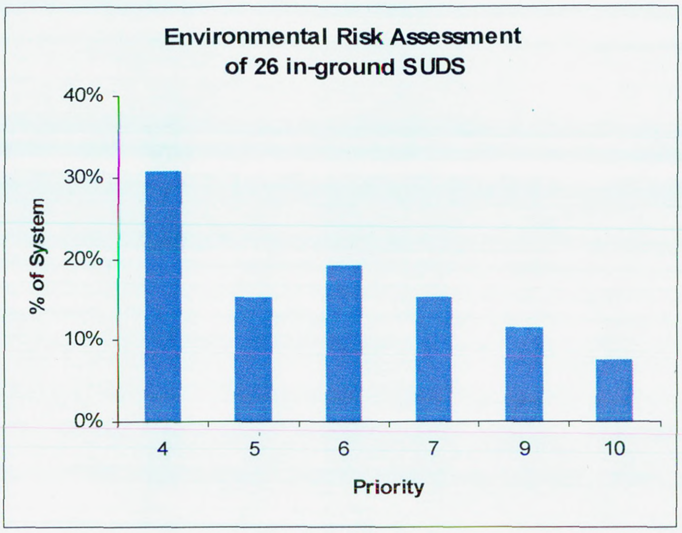



**Figure 4-5: Risk Assessment results**

It can be seen from this risk assessment that more than 30% of all system required immediate action to lower their risk of flooding or water quality pollution and these are associated with priority 4. Priority 5 to 6 was calculated for 35 % of all systems and these also required mitigation measures to be carried out. In general, these systems were associated with less frequent local flooding problems or less hazardous water quality pollution. No action was identified for 20% of all systems.

A comparison of this risk assessment and the Schlüter score is presented in Figure 4-6.

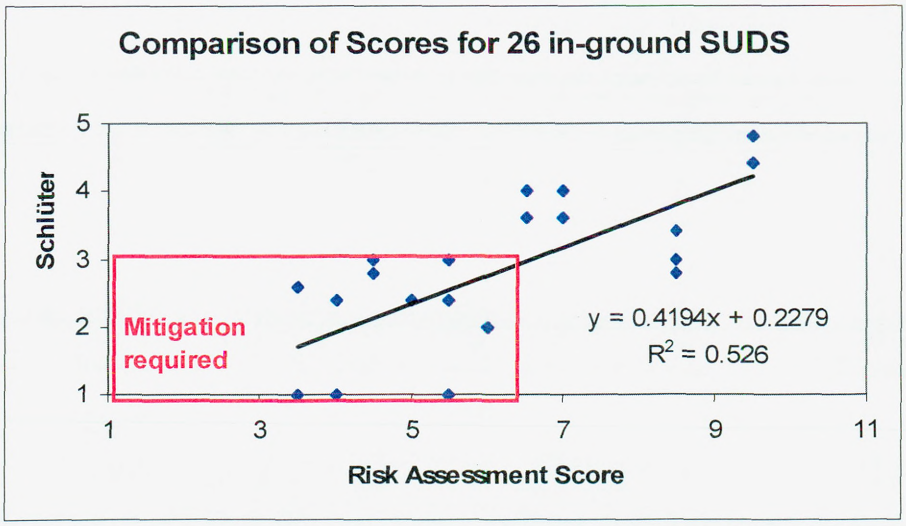



**Figure 4-6: Comparison of Risk Assessment and Schlüter score**

It can be seen that, whilst results are more scattered than expected, the overall trend of both scoring systems is comparable and an $R^2$ of 0.53 was achieved. The reason for the scattered results is mainly due to the difference in methodology. Whilst the Schlüter methodology produces scores from 1 to 5, the environmental risk assessment score is not expected to be less than 3. For example, pollution incidences of outflow containing high levels of heavy metals and hydrocarbons could occur from accidental spillages which would be of a probability less than once per 2 years (Severity 1 & Probability 3 = Priority 3). Similarly, property flooding from in-ground SUDS would be expected on rare occasions, only. Figure 4-6 also shows the systems which required mitigation measures to be undertaken to reduce their environmental risk and these received a Risk Assessment Score of 6 or less and a Schlüter score of 3 or less. There are two outliers which received a Schlüter score of 3 but do not impose an environmental risk.

This comparison offers confidence in the results produced by the Schlüter score as both scoring systems identified the same systems requiring mitigation measures and the overall trend is similar. It also shows that systems of Fair performance do not necessarily impose an immediate environmental risk. This will always depend on the location of the system (i.e. Greenfield) and the types and amount of expected pollutants and the type of receiving water course (i.e. sewer system, urban stream, highly sensitive water course).

## 4.5 Detailing examples of good practice

### 4.5.1 Excellent detailing, System No 32

Only one of the systems under investigation was found to be performing excellently and this one was System No 32. It is located at a small extension of a housing development in Broxden, Perth. The inlet pipe of the infiltration trench runs for the system's full length and this maximises the inflow capacity (see Figure 4-7 and Appendix B.5). The sump at Broxden is also 1.0m deep and the inlet and outlet pipes (of the inlet sump/ chamber) are level, promoting good debris settlement.

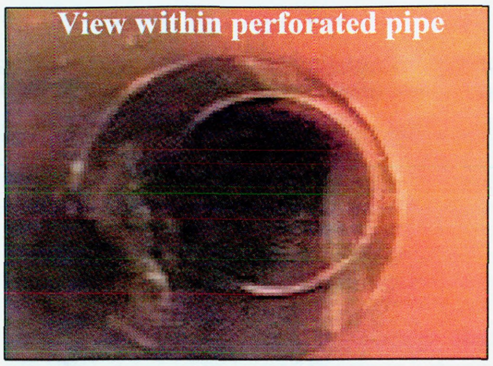



**Plate 4-1: Clean inlet to infiltration trench in Figure 4-7**

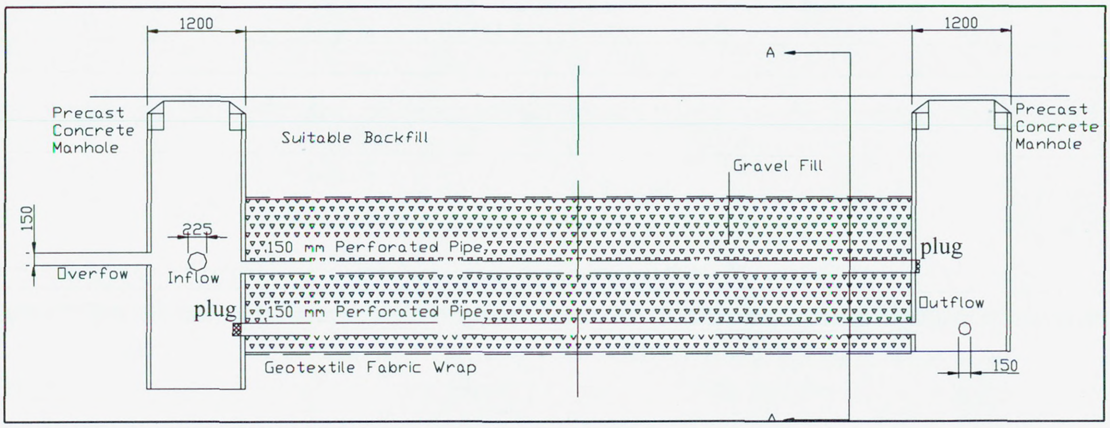



**Figure 4-7: Infiltration trench at Broxden**

Crucial for the performance of the system at Broxden are the two plugs, cutting-off flow at either end of the drainpipes. Their installation is essential for the operation and maintenance of the system. They can be removed for general inspection and to enable cleaning by high pressure jetting and flushing of drainpipes. The system is located along a relatively steep slope discharging to the Craigie Burn, which runs at a lower elevation. There is no risk of water ingress from the receiving water. However, there are a number of points, which could further improve the system at Broxden:

- **Installing a dip plate in the inlet:**

  *A dip plate would hold back any floating particles and chemicals and reduce the flow velocity to promote debris settlement.*

- **Raising the elevation of the outlet:**

  *Raising the elevation of the system outlet would promote infiltration and utilise additional storage (see Section 5.9).*

- **Raising the top perforated pipe:**

*An increase in distance between the two perforated pipes would improve water treatment by filtration. Raising the top perforated pipe would also activate the lost storage volume above the high-level perforated pipe.*

- **Increasing the elevation of the overflow:**

*The overflow could be elevated further to provide additional storage volume and reduce the number of overflow events.*

### 4.5.2 Good flow distribution, System No 29

An alternative soakaway arrangement is illustrated in Figure 4-8 and this shows an end-pipe system located at Hatton. It is also located downstream from a conventionally drained suburban development (of around 24 houses) where the outlet is to a very small watercourse. In this case there is a reasonable chance of longer term good behaviour, firstly, because the SUD system was shallow and not below the water table. Also, there are three flow routes from the inlet manhole into the filter material, providing good flow distribution and giving a reduced pollutant load per length of pipe. The elevation of the pipe maximises the system's storage as it is located close to the top of the filter material.

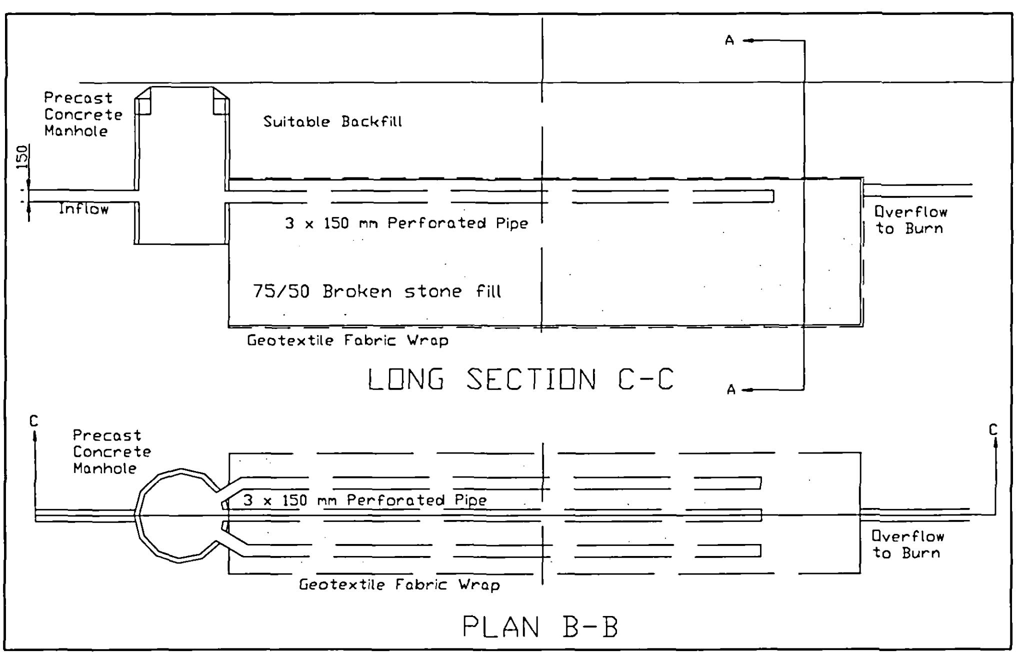
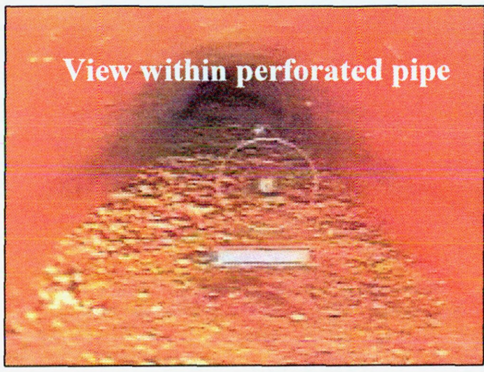



**Figure 4-8: Shallow soakaway arrangement**

The site visit to this location provided unexpected findings as the inlet sump of the system was filled with sediments and there was also a considerable amount of sediment in each perforated pipe (see Plate 4-2). Findings at this site show the typical impact of construction runoff and poor site management. Although both the design and construction of the system showed good practice, the system silted up quickly as it was connected to the surface drainage too early and sediment management was poor. Housing construction was ongoing and huge amounts of sediments had been mobilised and flushed into the soakaway system.



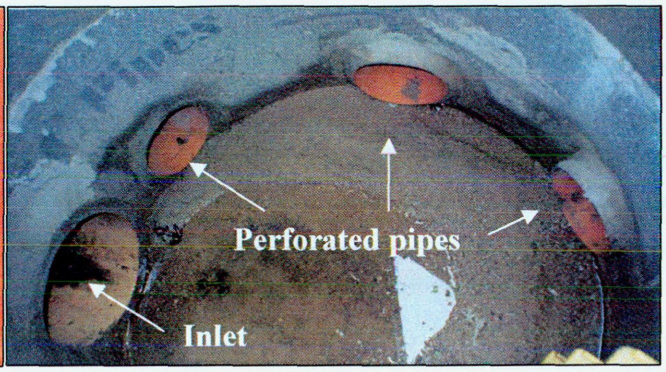




**Plate 4-2: Sedimentation at soakaway from Figure 4-8**

Unfortunately, this site lacked an inspection chamber downstream of the system, which would improve inspection and maintenance in the longer term.

### 4.5.3 Good flow control and well constructed, System No 11

The last example of good practice is another end-of-pipe-solution at Transy Estate Dunfermline. It is System No 11 and a long section including construction details is shown in Figure 4-9. The infiltration trench serves the roof and road surfaces of 12 houses. It is well constructed and showed good peak flow reduction and satisfactory flow volume reduction during the monitoring period of six months.

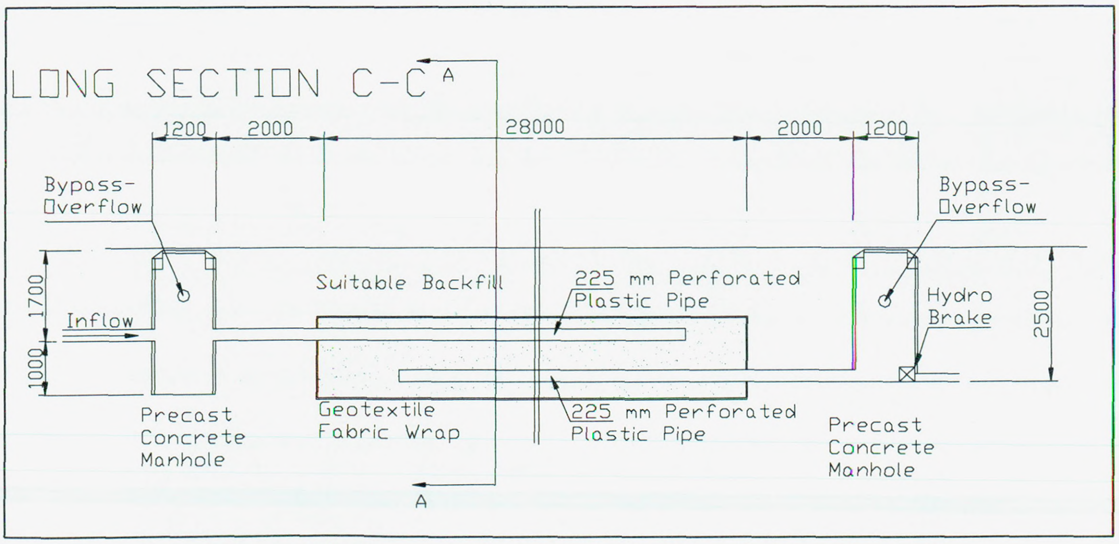



**Figure 4-9: Construction detail of infiltration trench detail.**

Plate 4-3 shows a view of the clean perforated pipe (a) and the upstream chamber (b).

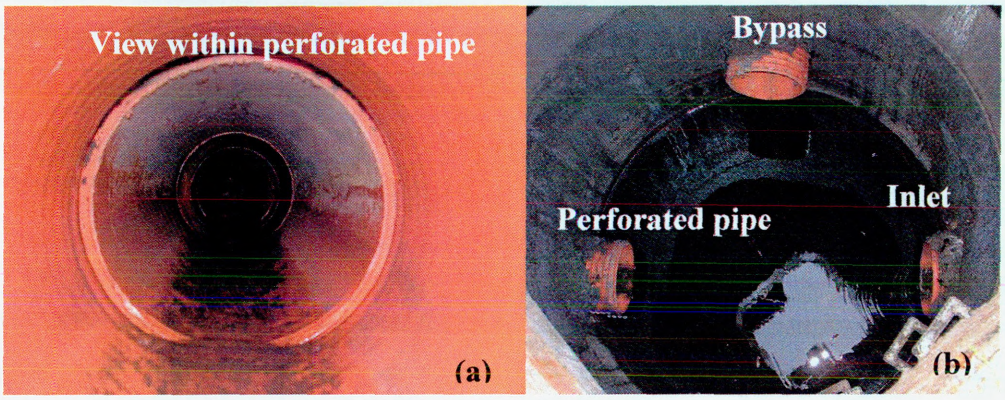



**Plate 4-3: Clean inlet into the infiltration trench**

The system uses three drainpipes to distribute the inflow into the filter material (see Appendix B.3). One low level outlet pipe discharges to the downstream inspection chamber and this contains a Hydrobrake flow control, which was installed to limit the maximum outflow to <10 $ls^{-1}$. There is a by-pass to prevent flooding in case of extreme events. Level recordings at the inlet showed that there was no surcharge during the monitoring period of six months. The inlet sump provides sufficient volume for debris settlement with a depth of one metre. Cleaning the perforated pipes would be difficult at this location as they are not easily accessed. The perforated pipes are not followed through to the inspection chambers and there are no rodding eyes for flushing through debris. Another drawback at this site was ground-water ingress, producing an increased outflow during long duration events.

## *4.6 Detailing examples of poor practice*

### 4.6.1 Insufficient sump capacity, System No 41

Figure 4-10 shows a long section of System No 41, which is an example of a recent in-ground SUDS construction, serving a small housing estate in Tullicoultry. This arrangement demonstrates poor design. The inflow pipe into the trench is located almost two metres below the chamber inlet and the sump provided is only 0.3 m deep. This arrangement promotes sediment flow into the trench as particles are kept in suspension. This is due to the excessive drop and small sump capacity.

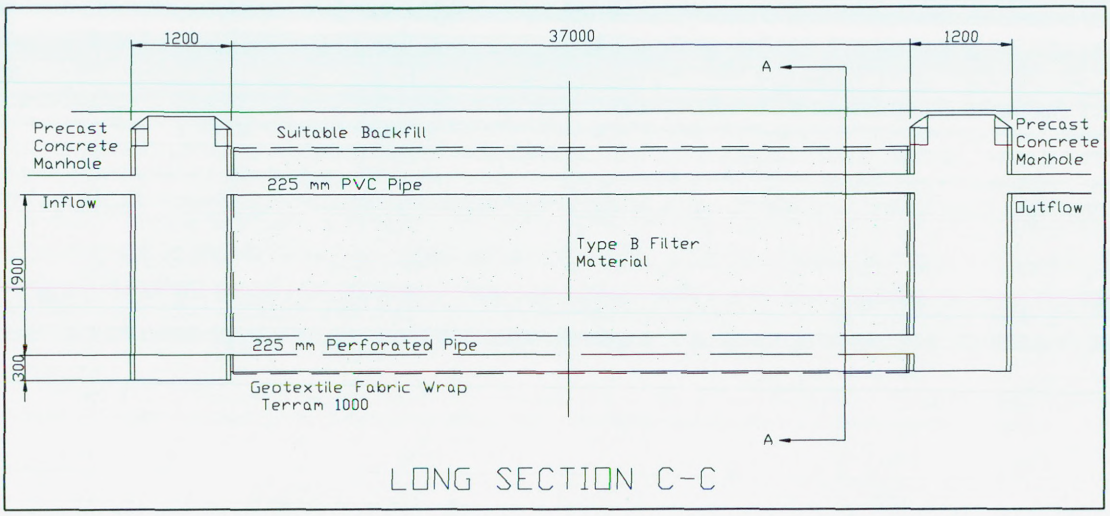



**Figure 4-10: Example of poor filter system arrangement**

Site inspection at this site showed sedimentation up to the top of the perforated pipe and this is shown in Plate 4-4. Although this site was developed very recently it is obviously designed with low confidence, using a high level by-pass and this is expected to operate frequently once the trench is completely blocked. This site is another example where runoff containing high sediment loads from housing construction is affecting the system's performance.

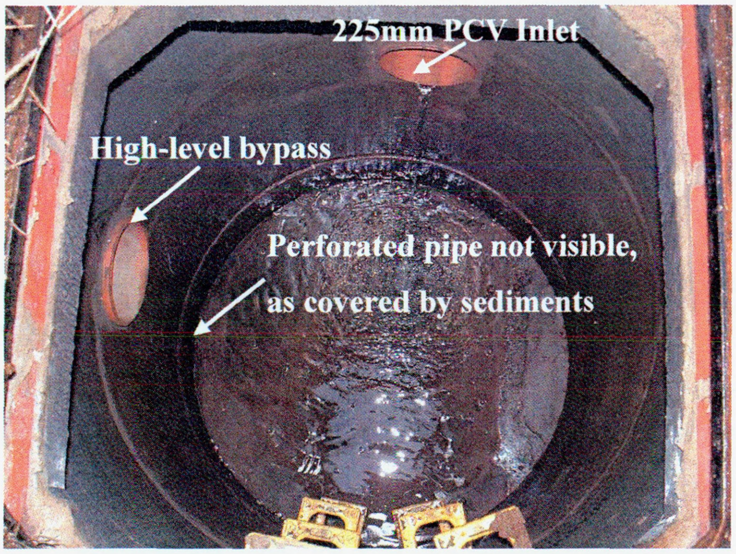



**Plate 4-4: Sedimentation at infiltration trench shown in Figure 4-10.**

### 4.6.2 Insufficient flow capacity, System No 1

The next example is the filter drain along Lang Stracht is System No 1. Hydraulic failure at this site is due to blocked inlets, which are via trapped gully pots discharging directly into the filter media. The flow capacity at this point is extremely low and small amounts of debris were found to have blocked the gully pot outlets. Plate 4-5 shows flooding of the system after an event of relatively small rainfall intensity of 13.2 mm/h and Figure 4-11 displays the detail of the gully pot connection with the filter drain. (See Appendix B.6 for information on the Lang Stracht site).

**Plate 4-5: Flooding at Lang Stracht due to blocked gully outlets**

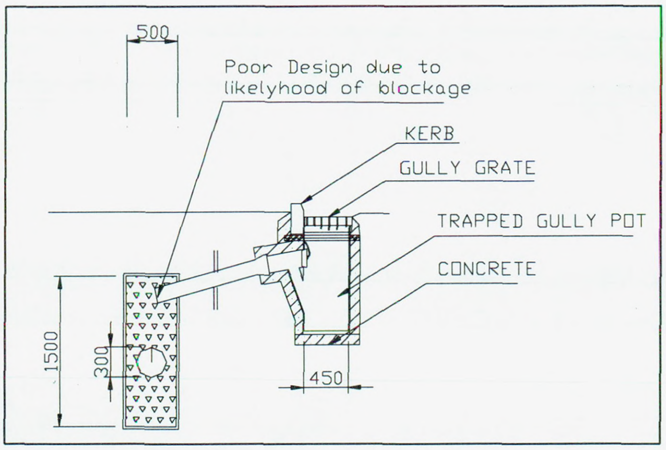



**Figure 4-11: Detail of gully pot connection to filter drain at Lang Stracht**

This detail has been improved at other locations with increased flow capacity and an overflow for extreme events and this detail is shown in Figure 4-12.

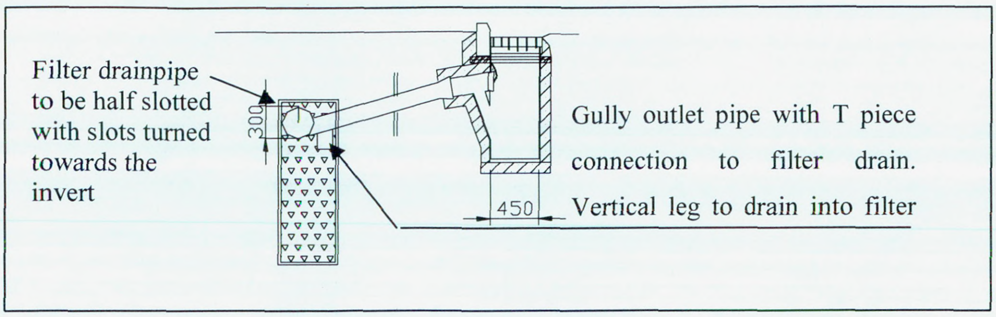



**Figure 4-12: Improved detail connecting gully pot outflow into a filter drain.**

### 4.6.3 Severe blockage, System No 12

The following example features an infiltration trench at Queens Gate Dunfermline (System No12). This system is located in a new medium-sized housing development of some 160 houses. The trench was designed to treat the first 10% of a 2-year storm event and was 52 m long, 2.2 m high and 2.2 m wide. Construction runoff with heavy sediment loadings blocked the inlet to the trench. Plate 4-6 illustrates the complete failure of this below ground SUDS installation.

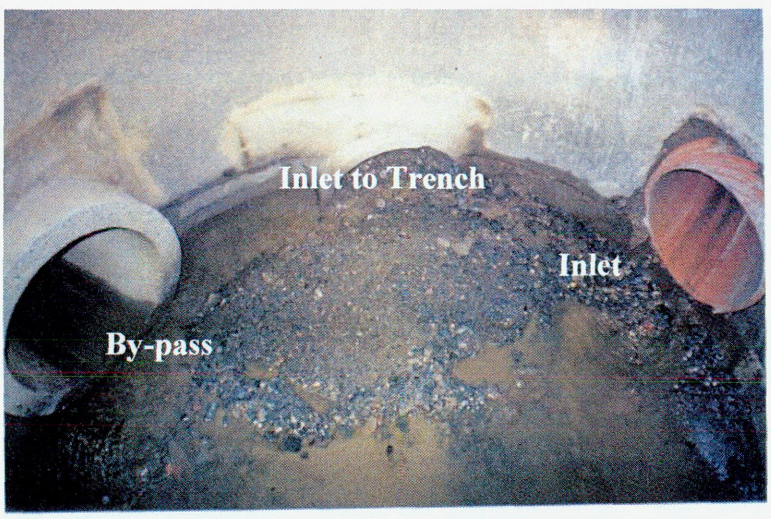



**Plate 4-6: Completely blocked Inlet**

In this case the concept is valid; the trench inlet has been set at a lower level than the by-pass so that low flows (with associated pollutant levels) will pass through the trench and have the opportunity to be filtered and to exfiltrate. The by-pass will come into operation at higher flows after the first flush has been discharged to the trench. Construction sediment has completely blocked the infiltration trench.

This example clearly illustrates unacceptable SUDS performance. However, it would be acceptable according to traditional drainage criteria since the total capacity to convey storm flows is unaffected, and surface flooding would be prevented except under extreme flows since the by-pass continues to operate. This is an example of hidden SUDS failure resulting from the ingress of construction debris.

An increase in sump volume for more sediment storage, in addition to the use of perforated pipes to distribute the inflow better, may have prevented the failure at this site. Unfortunately, no perforated pipes were installed and all debris and sediments accumulated at the trench inlet, blocking it very quickly. Also, construction practice should have included sediment management to prevent high sediment load entering the drainage system.

### 4.6.4 Lack of flow control, System No 2

One commonly encountered infiltration trench arrangement at the lowest point of a surface water installation is illustrated in Plate 4-7. The location is at the edge of a development of around 40 houses on a relatively cramped site. The SUD system adopted at this site is an arrangement of twin 400mm perforated pipes and was installed at an early stage of the implementation of the SUDS policy in Scotland. The installation was conceived in the belief that there may be some exfiltration through the perforations, but clearly this would be difficult since the stream (by the trees in Plate 4-7) is only slightly below ground level.

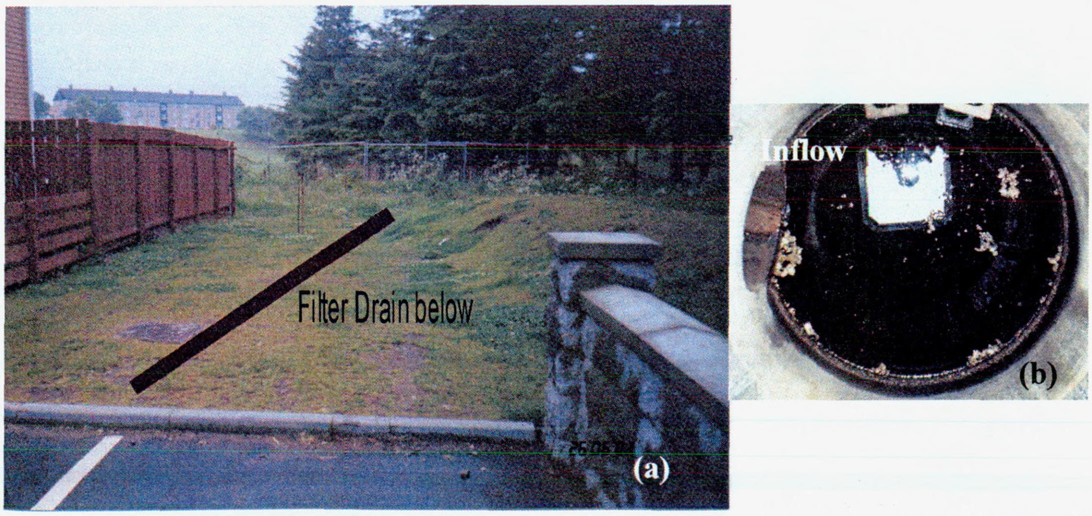



**Plate 4-7: Location of an infiltration trench (a); View into upstream chamber (b)**

In addition, soil conditions were such that there was little potential for infiltration. On-site inspection confirmed these findings of a high water table and poor exfiltration. Plate 4-7 (b) shows standing water within the inlet chamber, permanently surcharging the two drainpipes. This system offers no flow control and the only pollution retention is through sediment settlement.

Typical details of this type of arrangement are illustrated in Figure 4-13.

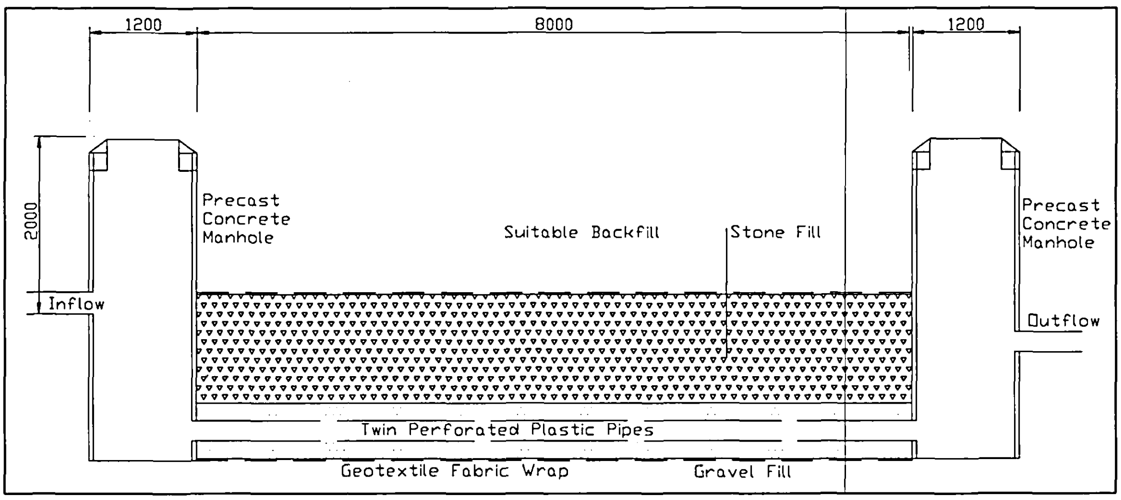



**Figure 4-13: Long section of a mature infiltration trench system**

## 4.6.5 Problems with filter drains & lateral inlets, Systems 5 and 15

A further commonly used technique is the French drain installed in car parking areas and roadsides. The car park shown in Figure 4-14 shows a site view of System No 5, which uses lateral inlets (or commonly known as offlet kerbs) connected directly to the free draining gravel strip with a perforated pipe or French drain below. It will be noted there has been some accumulation of leaves at the lateral inlets. Although most blockages were not complete, this is a very new installation and is typical of those found which may become problematic in future.

**Figure 4-14: Car park inlet using lateral inlets**

Another example was located along a section of Spine Road. System No 15 shows an inlet detail which is via slotted lateral inlets. Although this system should improve water quality performance due to the filtering out of pollutants, which is promoted by disconnecting the drainpipe from runoff, the following problems were discovered:

- Filter material was distributed onto the road (see Plate 4-8 (a))
- Blocked lateral inlets resulted in local flooding (see Plate 4-8 (b))
- Broken lateral inlets at three locations (see Plate 4-8 (c)


**Plate 4-8: Problems with offlet kerb arrangements along Spine Road**

### 4.6.6 Highly turbid outflow from typical road-side filter drain, System No 10

This site, which is another section along Spine Road in Dunfermline is characterised by extremely low traffic volume. The inlet of System No 10 is via typical trapped gully pots, which are connected into the perforated drainpipe. The system also receives sheet flow from the adjacent footpath. Appendix B.2 shows details of this filter drain and Plate 4-9 shows a site view. Turbidity monitoring of this system showed relatively high readings of up to 700 NTU. This was unexpected, considering the small amount of traffic and general cleanness of the road. It is thought that, since the gully pots are directly connected to the perforated drainpipes, very little pollutants are filtered out but are merely discharged downstream. Clearly this is unacceptable at locations where pollutant removal is a main objective. However, the hydraulic performance of this detail was satisfactory with an average volume reduction of just under 40%.

**Plate 4-9: Filter drain along Spine Road**

## *4.7 Discussion on detailing*

Forty-three different sites were inspected and their hydraulic and water quality performance was assessed on a visual basis. A CCTV survey was undertaken for a few selected sites. In addition, on-site monitoring was undertaken at six sites. Although it was difficult to generalise the findings, the following points are drawn.

The performance and longevity of many in-ground SUDS is impaired by high sediment loads from construction runoff (Plate 4-2, Plate 4-4 and Plate 4-6). This problem could easily be prevented by protecting the drainage inlets until the completion of construction and site clean-up and the drained area has stabilised. There are various techniques available to stop high sediment load entering storm drains and these are extensively used in the US. Information is given in USEPA (2002).

Some of the more mature sites were designed with low confidence in their long term performance and this has led to poor design. For example, throttles were not installed due to a lack of confidence in the ability of the throttle to operate effectively in the long term, or for fear of blockage. The result has been that the storage volume cannot be utilised effectively and these systems act merely as large storage tanks (Figure 4-10 and Figure 4-13).

The survey showed that disconnecting the system's inlet from the outlet provides better pollution retention in comparison with systems which discharge directly via perforated drainpipe. Sediments with associated pollutants are filtered out and retained within the filter medium rather then flushed through. However, the reduced hydraulic performance from these systems has to be taken into account. Frequent flooding was discovered at two sites due to the reduced hydraulic capacity (See Plate 4-5 and Plate 4-8).

A few sites were found which did not use any inspection chambers or rodding eyes and these are impossible to maintain and clean. Other sites could have been improved further by using additional features, such as a dip plate or a rodding eye or the modification of existing details. Often the volume of the sediment sump was not sufficient or the level of the perforated pipe was inappropriate, promoting sediment input into the trench.

Many sites use a high-level by-pass or an overflow. These sites impose the risk that failure due to blockage may never be noticed and the by-pass could be operating continuously (See Plate 4-4 and Plate 4-6). The installation of overflows has to be assessed on a site-by-site basis but many installations were found where overflows could have been saved, there was no risk of property flooding.

## 4.8 Maintenance of in-ground SUDS

### 4.8.1 Overall findings

From this investigation, the key maintenance issues were:

(1) No maintenance had been carried out at 39 of the locations.

(2) Maintenance in the form of removal and replacement of the filter material had been undertaken at two locations.

(3) Maintenance in the form of jetting had been undertaken at two locations (see Section 4.8.4)

In spite of this lack of maintenance, it was clear that maintenance was urgently required at many of the systems. Of the 32 for which performance and maintenance could be assessed, 17 (50%) required substantial work to be undertaken before the system could be considered to be operating satisfactorily and in a condition to be maintained regularly. As a result of this appraisal, it was decided that projected maintenance activities could not be based on maintenance schedules, but would have to rely on estimates of work requiring to be undertaken to restore the condition of the underground system and to maintain it in a satisfactory state. An inventory of the maintenance activities which were required at all of the sites was prepared, and from this inventory, the long- and short-term maintenance requirements were assessed (see Table 4-16 and Table 4-17).

Two categories of maintenance activities were required:

**One-off tasks** – to restore the SUDS to a satisfactory condition

**Ongoing tasks** – to maintain operation throughout the life of the system.

### 4.8.2 One-off maintenance tasks for filter drains

A significant number of in-ground filter drains, infiltration trenches and soakaways were found to require major upgrading before they could be considered to be satisfactory. Of the 43 sites examined, one-off maintenance tasks were required for 32, and Table 4-13 indicates the range of tasks required. The different tasks are considered in the following paragraphs.

| Description of tasks required | Number of Systems requiring tasks |
|---|---|
| Complete reconstruction including access chambers/ sumps | 5 |
| Substantial replacement of filter material and associated builder tasks | 7 |
| Cleaning out construction debris and jetting of perforated pipes | 5 |
| Inspections and minor cleaning | 15 |

**Table 4-13: One-off maintenance tasks for filter drains, infiltration trenches and soakaways**

### *Complete reconstruction*

Approaching 20% of the systems were found to require major works amounting to rebuilding before they could be considered as being acceptable and adopted by an operating authority. Only two causes were found for this category of failure;

Two systems were poorly conceived and key components were below the local water table. It was not possible even to envisage how they could be reinstated, so reconstruction was the only option. It is also worth noting that one of the systems had no flow control and would have required major works even were it to have been above the water table. It should be noted that these two systems were installed shortly after the introduction of the SUDS policy by SEPA and there was no track record of successful installations in Scotland. There was at the same time not insignificant resistance to SUDS in the area.

Three systems suffered from inappropriate site construction practice, which had led to the SUD being overwhelmed by construction material and debris. Such practices are not the norm on construction sites, but the damage at these three sites was so great that normal cleaning/ jetting procedures could not have produced satisfactory results.

### *Substantial replacement*

Approaching 25% of the systems required a significant amount of rebuilding. Again, there were two major causes:

- Poor site construction practice as described above. Although there were accumulations of sands and gravels, these were not considered to require the scale of rebuilding as above. However, the systems were clearly unable to operate as designed and the deficiencies were so great that they required attention prior to adoption.
- Minor inlet and filter drain installation. These tasks were not considered to be complex and could be undertaken by contractors using small equipment. However, reasonable lengths of drain were frequently involved and the work could not be considered to be minor. Inlet detailing required attention as a result of blockages and many inlets could not satisfactorily be cleaned due to poor detailing. At a number of locations this required the installation of a distributor pipe along the top of the trench.

*Cleaning and jetting*

After exclusion of those sites requiring more significant attention, approaching 20% of the systems required cleaning and jetting. These tasks were important as far as system operation was concerned, but were relatively mundane tasks involving low cost manhole entry and jetting out types of tasks.

*Inspection and minor cleaning*

Nearly half of all the systems inspected required little or no work done. It is assumed that an inspection would require to be undertaken prior to adoption. At the most, a few of the inlets might need to be cleared using drain rods, and litter/ polythene would need to be picked out of an outlet.

### 4.8.3 Ongoing routine maintenance tasks for in-ground SUDS

Ongoing maintenance tasks have been categorized into the tasks to be undertaken, and the frequency with which they are likely to be required. Table 4-14 shows the numbers of systems requiring the different ongoing maintenance tasks.

| Task | Systems |
|---|---|
| Inspections by supervisory staff only | 33% |
| Gully pot emptying | 33%* |
| Jetting, minor cleaning and sump emptying, tasks vary depending on design | 33%* |
| Removal of top layer and cleaning of filter media – heavily used roads | 20% |
| * A number of sites required both tasks | |

**Table 4-14: Ongoing maintenance tasks for investigated in-ground SUDS**

The only 'hard' data available were from highways operators who undertook routine inspections and responded to emergent problems with a preventative maintenance programme. Otherwise, the maintenance programme and frequencies proposed were based on informed estimates of the likely accumulation of sediment and contaminants. The different tasks are outlined in the sections below.

### *Inspections*

It has been noted that all properly funded maintenance organisations carry out routine annual inspections. These are carried out by experienced, trained personnel.

### *Gully pot emptying*

More than three quarters of all the sites studied used trapped gully pots for the entry of water from road surfaces into the filter material. A number of instances of blockage of the entry from the gully pot into the filter material were noted and it is essential that this risk is minimised by gully pot emptying. This is a standard activity for most roads authorities and is undertaken from above ground.

### *Jetting, minor cleaning and sump emptying*

These activities require some form of manhole entry and removal of contaminated sediment and water. Most systems incorporate at least one perforated pipe in the filter material and an inlet manhole that should incorporate a sediment sump.

**Jetting** – of the perforated pipe(s) running the length of the systems. Leaves and fine material require to be washed through. The downstream manhole requires to be blocked to prevent the washwater from reaching the watercourse. This would be done by plugging the outlet and pumping out the contaminated washwater.

**Sump emptying** – sediment removal and subsequent disposal.

**Minor cleaning tasks** – normally construction debris. This is only significant because it is below ground.

### *Removal of top layer and cleaning of filter media*

One particular maintenance technique was found to be used on the filter drains alongside major roads. To ensure quick turn around on site (and to minimise costs), all filter media down to but not including the bottom perforated drainpipe is excavated. The media is removed from site for washing or disposal, and new media is installed. This approach might be applied in more densely populated areas, but it has a number of drawbacks;

- Pollutants that had accumulated in the section below the perforated pipe continue to seep into the adjacent soil subsequently into the ground water.
- Gullies or other point inputs would easily be damaged making re-instatement necessary.

- The top perforated pipe (recommended in many locations) would require to be replaced.

An alternative method to replacement of gravel is used for roadside filter drains which receive lateral sheet flow (see system in section 3.3.3). At this type of system, blockage is often caused by sedimentation at the top of the gravel. To rectify this problem and promote water inflow into the gravel, a rake dozer is used to uplift and disturb the top layer of gravel. This type of maintenance is the easiest and most rapid cleaning technique. However, the drawback of pollutant accumulation remains.

### *Frequency of ongoing maintenance for filter drains*

The systems inspected were relatively young as was noted in section 4.3 Consequently the full range of maintenance activities required could not be observed and estimates of the frequencies had to be made. The resulting frequencies are shown in Table 4-15.

| Description of tasks required | No | Frequency |
|---|---|---|
| Inspections by supervisory staff only | All | Annually |
| Gully pot emptying | 30% | Annually |
| Minor works - jetting, cleaning and sump emptying, tasks vary depending on design | 60% | 10 Years |
| Minor works - jetting, cleaning and sump emptying, tasks vary depending on design | 15% | 5 Years |
| Removal of top layer and cleaning of filter media | Heavily used roads | As required |

**Table 4-15: Likely frequency of maintenance activities on in-ground SUDS**

| Number | Location | Site | One off tasks required | Continuing maintenance needs |
|---|---|---|---|---|
| 1 | Aberdeen | Lang Stracht | Excavate top soil and gravel layer. Install high-level perforated pipe with Y pieces at each gully. | Gully emptying. Jetting of top and bottom perforated pipes. |
| 2 | Aberdeen | Woodend | Complete reconstruction before this system could be adopted | |
| 3 | Aberdeen | Arnhall Business Park | This is a road section. Normal gully emptying needed | Normal gully emptying needed |
| 5 | Aberdeen | Arnhall Business Park | Lateral inlets require to be removed and inlet arrangements changed. | Car park system |
| 7 | Aberdeen | Walker dam | None | Empty sump with suction machine. High pressure jetting of inlet perforated pipe. Pumping out wash water. Gully emptying once a year |
| 8 | Blairhall | Housing Estate | None | Relatively infrequent desilting at inlet. Gully emptying once a year. |
| 9 | Dundee | Swallow Roundabout | Complete reconstruction should be undertaken as part of the routine maintenance. i.e. routine maintenance means regular reconstruction. Normally the drain should also be replaced, otherwise pollutants will not be cleaned. Gully emptying every 6 months | |
| 10 | Dunfermline | Spine Rd, DEX | None | Jetting and gully emptying. Road not very busy. Gully emptying once a year |
| 11 | Dunfermline | Transy Estate | Install rodding eyes on perforated pipe system | Jetting every 10 years and gully emptying annually |
| 12 | Dunfermline | Queens Gate | Removal of construction sediment from inlet in the system. This will involve some excavation and replacement of perforated pipe | Jetting every 5 years & gully pot emptying annually |
| 13 | Dunfermline | Woodmill Road | Gully outlets are a good detail. Some cleaning out of debris needed. | Jetting every 5 years & gully pot emptying annually |
| 14 | Dunfermline | Tesco Car park, DEX | Reprofile the ground upstream to prevent sediment washing off from adjacent land. | Periodic removal of gravel from top of drain. |
| 15 | Dunfermline | Spine Rd, DEX | Reconstruct drain including kerbs. This system is not a success because the lateral inlets lead directly into the gravel. This cannot be cleaned. | Badly conceived system |
| 17 | Edinburgh | Forth Road Bridge North | Complete reconstruction should be undertaken as part of the routine maintenance. i.e. routine maintenance means regular reconstruction (every 2years). Normally the drain should also be replaced, otherwise pollutants will not be cleaned | |

**Table 4-16: Infiltration systems maintenance appraisal**

| Number | Location | Site | One off tasks required | Continuing maintenance needs |
|---|---|---|---|---|
| 18 | Edinburgh | Forth Road Bridge South | Complete reconstruction should be undertaken as part of the routine maintenance. i.e. routine maintenance means regular reconstruction (every 2years). Normally the drain should also be replaced, otherwise pollutants will not be cleaned | |
| 20 | Falkirk | Wallacelea Rumford | Remove construction debris. | Jetting |
| 23 | Falkirk | Business Park, Larbert | Minor removal of debris | Minimal long-term maintenance required since site is not heavily used. |
| 25 | Falkirk | Queens Drive Larbert | Construct inspection chamber and possibly remedy other details | Cannot be assessed |
| 27 | Falkirk | Taymouth Rd, Heatherlea | Remove sediment from sediment traps, clean gully pots | Minimal long-term maintenance required since site is not heavily used. |
| 28 | Findochty | Netherton Farm | Reconstruct - system badly conceived due to high water table | _ |
| 29 | Hatton | Lairds Park, Fintray | Remove construction debris and jet perforated pipe to restore system up to design standard. | Minimal long-term maintenance required since site is not heavily used. |
| 30 | Invergowrie | Invergowrie Mill | _ | _ |
| 32 | Perth | Broxden | Minimal required | Minimal long-term maintenance required. |
| 33 | Perth | Glencarse A90 | Main road filter drain. No immediate action | Top layer of gravel to be removed for cleaning. |
| 34 | Perth | St Madoes Old | Install sediment trap. | Minimal long-term maintenance required. |
| 35 | Perth | St Madoes New | Completely rebuild system which has become blocked due to construction debris | Cannot be assessed |
| 37 | Tayport | Sandy Hill | Investigate cause of contaminated outflow from one of the filter drains | Minimal long-term maintenance required since site is not heavily used. |
| 38 | Tayport | Sandy Hill, S1 | Minimal required | Minimal long-term maintenance required since site is not heavily used. |
| 39 | Tayport | Sandy Hill, S8 | Minimal required | Minimal long-term maintenance required since site is not heavily used. |
| 41 | Tullicoultry | Fir Park | Completely rebuild system which has become blocked due to construction debris. This must include construction of a deeper inlet sump | Minimal long-term maintenance required since site is not heavily used. |
| 42 | Westhill | Leddach Farm New | Minimal required | Minimal long-term maintenance required since site is not heavily used. |
| 43 | Westhill | Leddach Farm Old | Cleaning out of construction debris and jetting. | Minimal long-term maintenance required since site is not heavily used. |

**Table 4-17: Infiltration systems maintenance appraisal (continued from Table 4-16)**

### 4.8.4 Maintenance undertaken at selected sites

*Maintenance of infiltration trench at Walker Dam*

Maintenance at Walker Dam (see Section 3.3.4 for site information) resulted in an improvement of the system's hydraulic performance. Figure 4-15 shows level and flow readings in the inlet to the upstream chamber. A significant hydraulic improvement can be seen after system cleanout.

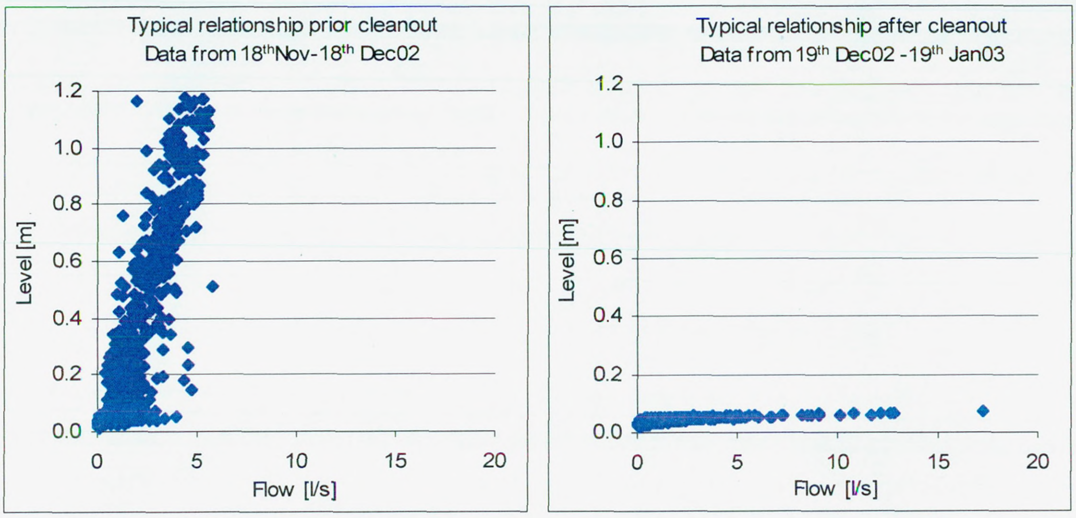



**Figure 4-15: Comparison of level data prior and after cleanout**

High pressure jetting and a chamber clean-out led to an increase in flow capacity and no surcharge of the upstream chamber was recorded afterwards. Plate 4-10 shows a view into the inspection chamber during maintenance.

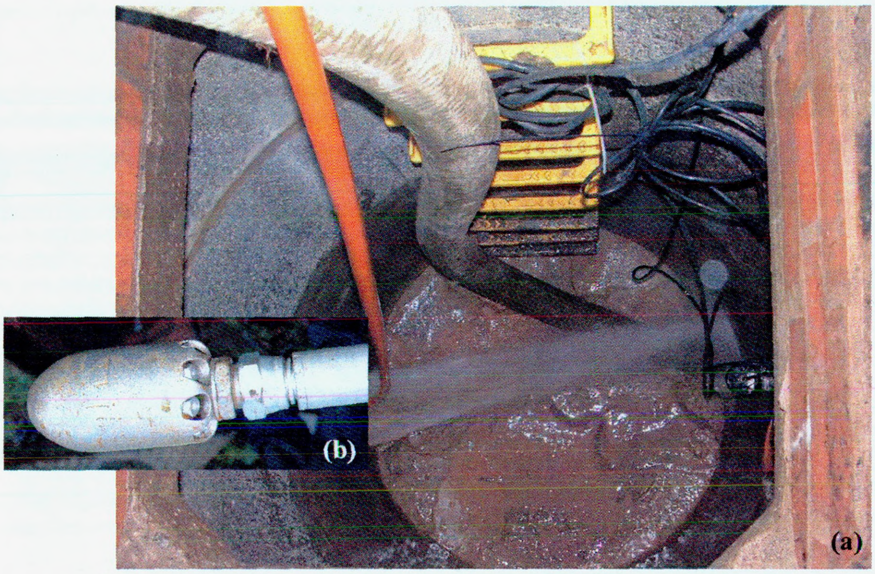



**Plate 4-10: Cleanout of chamber at Walker Dam (a); High pressure head (b)**

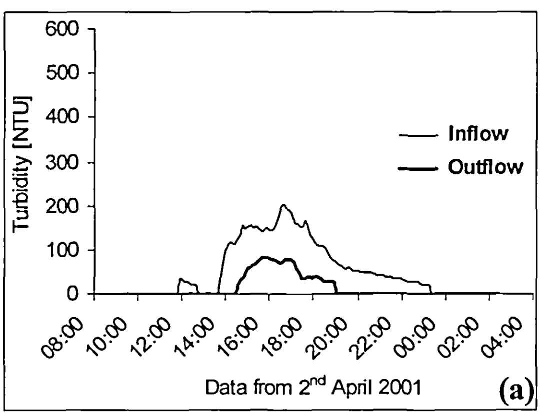
There was also a recorded improvement in flow reduction, when comparing data prior and post cleanout. The reduction of flow by volume improved from 23% prior, to over 40%, post cleanout.

However, the fact that fine particles had accumulated in the downstream chamber indicates that not all pollutants could be retained within the filter medium. Plate 4-11 shows the highly turbid outflow when slightly disturbed. Although the cleanout had a positive effect on the hydraulic performance, it is suspected that much of the storage volume is lost due to sediment accumulation and this could not be recovered without system replacement.

**Plate 4-11: Highly turbid outflow from Walker Dam**

*Maintenance of roadside filter drain along Lang Stracht*

Maintenance at Lang Stracht (see Section 3.3.1) initiated sediment movement and a highly turbid outflow was monitored thereafter (see Section 3.5 for information on monitoring instruments). Turbidity monitoring continued for three months after gully pot cleanout confirming high turbidity readings. It is thought that high pressure jetting of the gully pot pipes mobilised sediments, which had accumulated in the gully outlets and the filter media. This resulted in a significant increase in turbidity (>1000 NTU) recorded at the system's outlet. Figure 4-16 shows typical turbidity readings at Lang Stracht prior and post cleanout and Appendix B.2 displays a detailed construction drawing. The turbidity readings at the inlet did not change significantly during the monitoring period but a significant increase was recorded at the outlet.



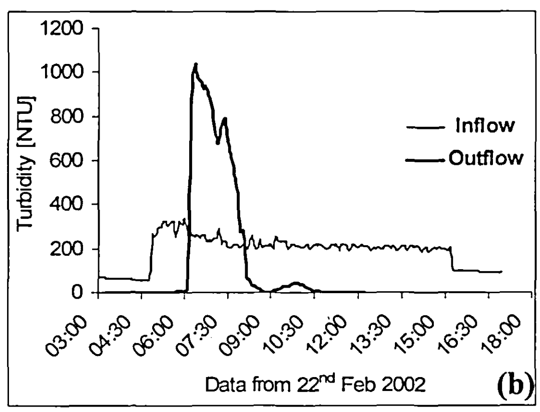




**Figure 4-16: Typical turbidity readings at Lang Stracht (a) prior cleanout, (b) post cleanout**

Sediment breakthrough at the system's outlet was also confirmed by on-site observation and this is shown in Plate 4-12.

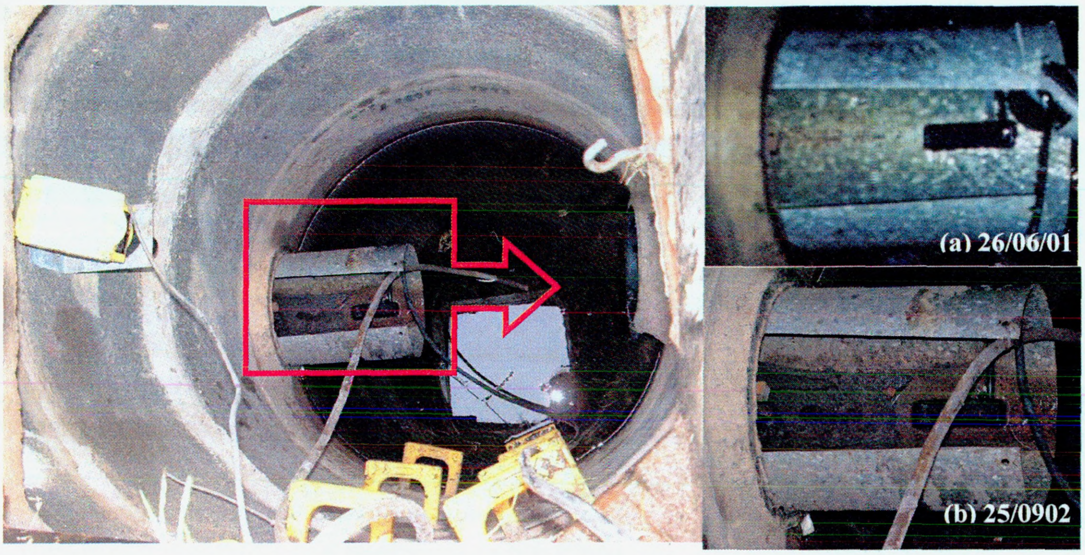



**Plate 4-12: Sediment breakthrough at the system's outlet (a) prior to cleanout (b) post cleanout**

No hydraulic improvement of the system performance was recorded after maintenance. Hydraulic analyses included periodic event investigation, and no significant change in hydraulic behaviour attributable to the cleanout was found. System monitoring started shortly after the system was constructed and this implies that the system had never operated satisfactory. The connection of the gully outflow does not provide enough capacity to discharge road runoff from the connected areas. Gully pots are located in intervals from 20 to 50 metres draining approximately 250 $m^2$ and this is sufficient for traditional drainage systems but not for this SUD system.

Monthly gully pot inspections were undertaken at Lang Stracht over a period of more than one year. Although this site did not suffer from runoff from unstabilised areas or construction runoff, it was found that gully pot sumps needed cleaning after 6 months.

### *Maintenance of filter drains along major roads*

Maintenance programs of filter drains along major trunk roads in the East of Scotland (System No 17, 18 and 33) is undertaken also on an incident basis. However, the operators experience was that removal of filter material has to be undertaken approximately every two years and inspection teams are continuously checking the system's performance. Once a system is blocked, filter material removal is undertaken using a JCB digger to extract all filter material above the drainpipe and replace it with recycled or new filter material. This operation is shown in Plate 4-13 (a) and (b).

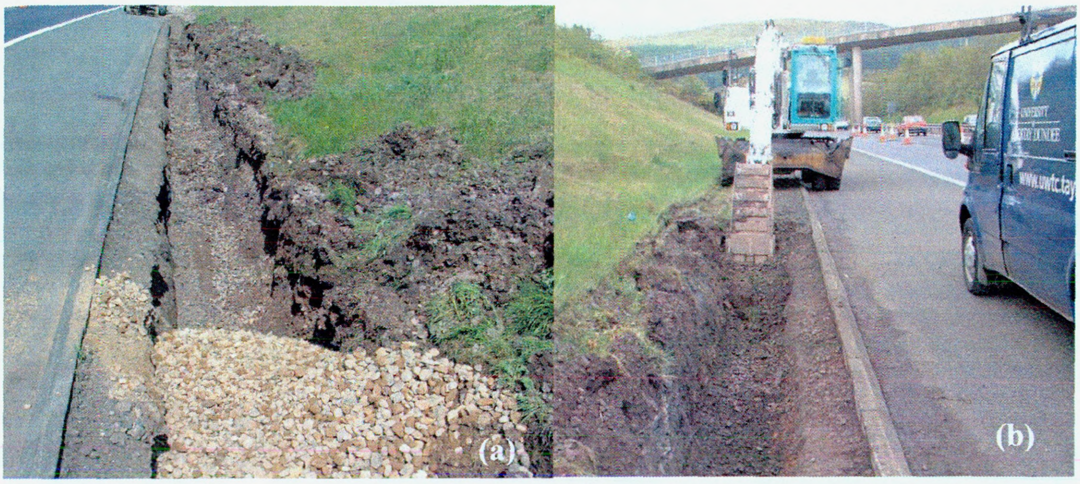



**Plate 4-13: Replacement of filter material (a); Extraction of filter material (b)**

Unfortunately, this operation does not remove any pollutants from the bottom section of the trench (below the drainpipe) but only improves the systems hydraulic performance (as noted earlier). In some locations the operator chooses to uplift the filter material using a rake dozer. This will create new flow paths to improve the hydraulic performance. Again water quality issues are not addressed with this technique with only local flooding problems are rectified.

## *4.9 Discussion of maintenance issues*

No maintenance programs were in place at most of the monitoring locations, which is due to staff and infrastructure limitations. Maintenance is carried out on an incident basis, which is sufficient for traditional storm water drainage but is a flawed approach for in-ground SUDS. In many cases, blocked in-ground SUDS cannot be cleaned and replacement is the only approach. Maintenance should be undertaken regularly and this may comprise of drain and gully pot cleaning and sediment chamber inspections. The maintenance intervals are site dependent and these may vary from twice per year up to once every ten years.

This survey showed that roadside filter drains may impose a long-term pollution risk to receiving waters. Maintenance is undertaken with the primary objective to enable hydraulic performance and it is thought that current maintenance techniques allow a great amount of pollution to accumulate within the system (see Section 4.8.4). To date there is no effective way to extract pollutants from the filter material and once blockage occurs, the whole system would have to be replaced. For major trunk roads, replacement is expected every two years to maintain hydraulic performance. The filter material is replaced only above the drainpipe (see Plate 4-13). The drainpipe stays in place and the filter material below the drainpipe is not replaced either. Pollutants which accumulate at the trench bottom contain the highest pollutant concentration and may impose a long-term risk in from of ground water contamination.

A number of sites were found where sediments and associated pollutants had travelled through the filter medium and had accumulated in the downstream chamber (See Plate 4-11 and Plate 4-12). At locations were pollution reduction is crucial for the receiving watercourse, system replacement has to be considered. To date there is no procedure to clean or remove sediments from the filter material.

Cleaning techniques were unsuitable for typically trapped gully pots, which discharge directly into the filter material of filter trenches (see Section 4.8.4). This included high pressure flushing of the outlet, resulting in mobilising of accumulated particles and extremely high turbidity readings at the system outlet (found at Lang Stracht).

# CHAPTER 5 PERFORMANCE EVALUATION USING A STANDARD MODELLING PROGRAM

*A brief review of standard software package for hydraulic simulation and their suitability for modelling flow through SUDS was presented in section **2.5**. The most suitable program (Erwin) is presented here in more detail, including various parameters which were used and how the model was built. Individual models are developed for each of the monitored systems and results from data analysis, including model calibration, are shown in Appendix G. A system comparison and overall findings from the computer simulation are outlined at the end of this chapter.*

## 5.1 Rationale

Individual models for each system were developed using a standard software program to improve the understanding of the system's hydraulic behaviour and its sensitivity to separate parameters. The computer simulation enabled a direct comparison between the different systems using one set of rainfall data in addition to design rainfall.

## 5.2 Methodology

A review of standard modelling programs is included in section 2.5. The software package Erwin was available to develop individual models for all sites under investigation and previous work by Schlüter and Jefferies (2002) showed successful computation of outflow from a pervious pavement when using Erwin.

Prior to model development, the type of catchment area element had to be selected and this was undertaken by comparing the three available elements in Erwin. Individual models were then built by inputting the physical dimensions and their site-specific head-discharge relationships (see Section 3.6.4 and Appendix C). Each model was as simple as practicable to gain the best understanding of the hydraulic behaviour and to gain an optimal fit to recorded flow data. Measured rainfall data had to be exported and translated into the mass data format which is used in Erwin. This was automated using a batch file.

A sensitivity analysis was carried out for the surface runoff element and this procedure identified the most important parameters to predict surface runoff. Each system was then calibrated using monitored data. This was undertaken using a numerical method for flow volume per event, as well as an eyeball fit for the hydrographs.

A design rainfall event and a selected period of time series rainfall data were both used to compare the performance between the systems.

## 5.3 The modelling program Erwin

This section describes the elements, including their specifications and the influence of each parameter used in Erwin. The importance of the catchment area element is shown and a short comparison of the surface runoff elements is outlined.

Figure 5-1 shows an infiltration trench system as an example, displayed in Erwin.

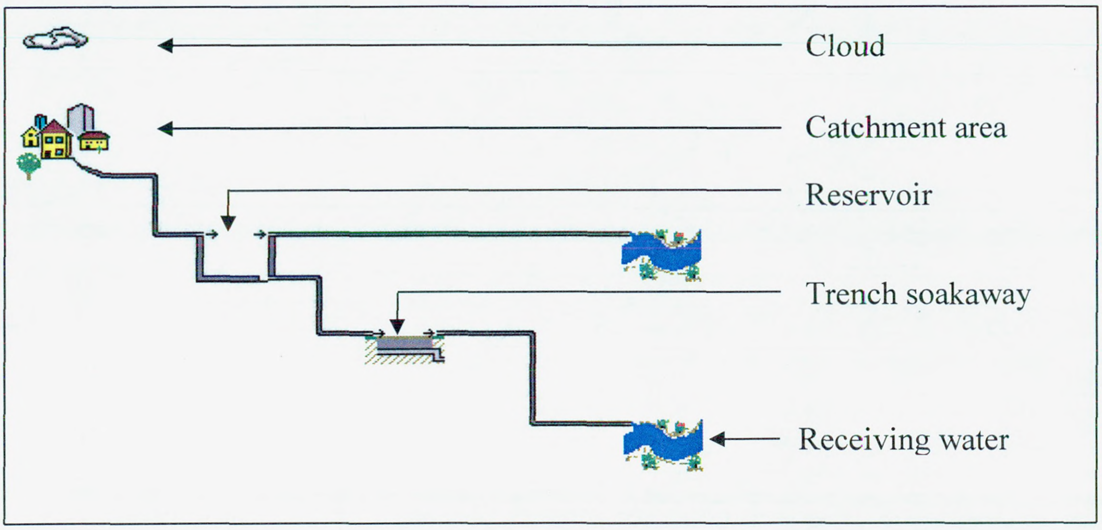



**Figure 5-1: Erwin model to simulate outflow from an infiltration trench**

The main elements of this model are the catchment area element and the trench soakaway element. The trench soakaway element represents the infiltration trench and the catchment area element incorporates values to imitate the system's catchment. This system uses a reservoir element which represents the upstream monitoring manhole and this incorporates an overflow.

The soakaway contains a perforated pipe, which conveys the water to the receiving water. Erwin allows exporting flow data at any point of the model and this can be used for calibration purposes.

## 5.4 Core modelling elements

All standard values given in this section are obtained from the Erwin Design Manual (Ingenieurgesellschaft fuer Stadthydrology, 1997).

### *The cloud element*

The cloud element incorporates on-site recorded rain data. These data were available in two-minute intervals in ASCII format. Erwin uses mass data format which is not compatible with ASCII format. A translation software was designed to transfer the recorded rain data from ASCII to Mass Data Format (Ingenieurgesellschaft fuer Stadthydrology, 1997).

### *Evaporation losses*

A standard evaporation loss model is incorporated in Erwin. Evaporation losses are taken into account using daily and annually variations and these represent average evaporation losses of North/ Central Germany. Erwin allows factoring these losses to adjust for other locations. However, no evaporation-loss data was available for any of the monitoring sites and the default value as incorporated in Erwin was used. Figure 5-2 shows the average daily evaporation and the hourly factor.

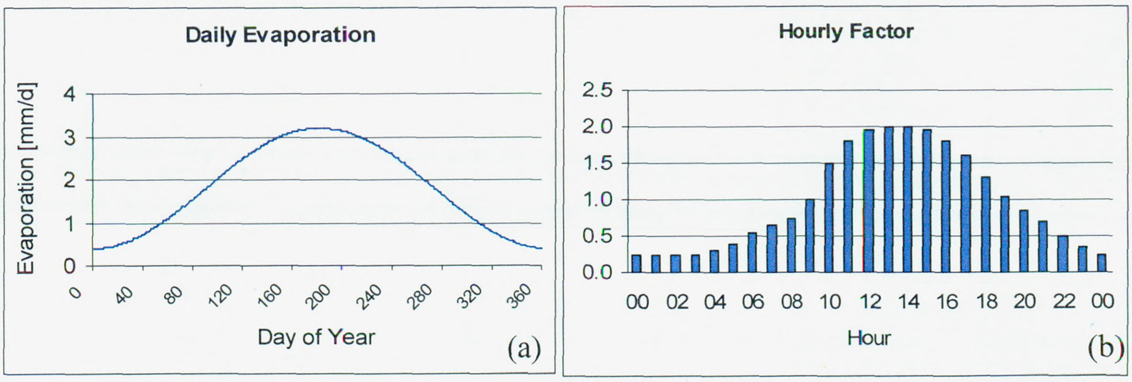



**Figure 5-2: Evaporation model incorporate in Erwin (Ingenieurgesellschaft fuer Stadthydrology, 1997).**

### *The catchment area element*

The catchment area element is most important for the runoff process. It incorporates values to represent initial and continuous rainfall losses, which are described in the following section;

Initial losses are deducted from the total rainfall up to a threshold value. With increasing rainfall, catchment surfaces are wetted and moisture losses reduced.

Losses due to depression storage occur after initial losses. When the threshold value is exceeded the depression storage is filled, additional rainfall is available for runoff.
The initial runoff coefficient describes the amount of runoff, which occurs after initial loss, without accounting of depression storage losses. In practise this would be runoff which occurs while depression filling is occurring.
The final runoff coefficient describes the amount of rainfall contributing to runoff after deducting initial losses and losses due to depression storage. At this stage only continuous losses occur which are taken account of by the final runoff coefficient.
Linear storage cascades can be utilised to attenuate the runoff hydrograph by means of two parameters: n & k. n is the number of storage cascades and k is the lag time within each cascade. This parameter has no impact on the event volume but it is used to attenuate the hydrograph and reduce the peak flow. Table 5-1 gives details of values for runoff from impermeable surfaces

| Variable | Value | Unit | Comment |
|---|---|---|---|
| Area | - | [m$^2$] | Site Specific |
| Initial Loss | 0.5 | [mm] | Calibration parameter |
| Depression Storage Loss | 1.5-2.3 | [mm] | Calibration parameter |
| Initial Runoff Coefficient | 20-30 | [%] | Calibration parameter |
| Final Runoff Coefficient | 75-95 | [%] | Calibration parameter |
| Linear Storage Cascade Parameters: n & k | 2*2=4 | [min] | Calibration parameter to attenuate flow |

**Table 5-1: Catchment area element data**

Value ranges are taken from Erwin User Manual (Ingenieurgesellschaft fuer Stadthydrology, 1997) and these were used to calibrate each site.

### *The trench soakaway element*

The parameters, which specify the trench soakaway, such as length, width, depth and characteristics of the drainage pipe, were obtained from construction plans and on-site measurements.
The infiltration rate was used as a calibration parameter as no information on the soil permeability was available.

The porosity of fill material determines the coarse pore content available for water storage within the sub-base of the gravel fill material. Table 5-2 gives values of the trench soakaway element.

| Variable | Value | Unit | Comment |
|---|---|---|---|
| Infiltration rate | - | [$ms^{-1}$] | Site Specific /Calibrated Value |
| Width | - | [m] | Site Specific |
| Length | - | [m] | Site Specific |
| Depth | - | [m] | Site Specific |
| Porosity of fill material | 25-40 | [%] | Calibrated Value |
| Pipe diameter | - | [mm] | Site Specific |
| Depth of pipe above ground | - | [m] | Site Specific |
| Head-discharge relationship | - | | Monitored values |

**Table 5-2: Trench Soakaway data**

### *The reservoir element*

The reservoir element is used for any storage element, but in this modelling it represents the monitoring manhole. It incorporates the flow-depth relationship obtained from the on-site monitoring. Data relating to the size of the monitoring manhole was obtained from on-site measurements and construction plans and parameters are given in Table 5-3.

| Variable | Value | Unit | Comment |
|---|---|---|---|
| Shape | Circular/ User defined | [-] | Site Specific |
| Depth | - | [m] | Site Specific |
| Radius | - | [m] | Site Specific |
| Head-discharge relationship | | [m] | Monitored values |

**Table 5-3: Reservoir data**

## 5.5 Comparison of runoff modelling elements

Erwin has three different elements to model the rainfall-runoff process. A comparison of these elements, which are roof, road and catchment area element was undertaken. The catchment at Broxden was used as an example and the results are presented here.

The roof element is the simplest of the three. It is used to model runoff from steep impermeable surfaces, which have a quick response to rainfall. The only input parameters are area, initial loss and a choice of whether water utilisation should be enabled or not.

The road element is mainly used to model runoff from flat impermeable surfaces. It enables a much better modification of the runoff process than the roof element, as it separates between initial and final runoff coefficients. Losses are taken into account using initial loss and depression storage. When modelling single events the degree of depression filling parameters can be used to allow for the initial wetness of the system.

The catchment area element is the most complex of the three and is mainly used for larger catchments. Most important for modelling large catchments is to separate between impermeable and permeable surfaces, as these result in significant differences in runoff behaviour. The cachment area element enables separating between permeable surfaces in addition to impermeable surfaces. Four coefficients are used to specify the permeable area; Wet and dry weather decay index; and initial and final infiltration rates.

Another important feature of the catchment area element is that it allows modification of the runoff curve. A linear storage cascade modelling approach is used. Two parameters, n and k, alter the peak of the hydrograph without affecting the total outflow volume. n is the number of reservoirs and k the detention constant in minutes. Multiplying those two parameters gives the total detention or lag time of the linear storage cascade (see Table 5-1).

## 5.6 Results

The catchment at Broxden was used as an example for comparing the three different surface runoff elements provided in Erwin. Initially, several test runs were undertaken with varying parameters, such as initial loss and percentage runoff to try and match the inflow hydrograph. The values that provided the best fit in comparison with the inflow hydrograph were kept constant in the road and catchment area element, to enable a direct comparison. The only value specified in the roof element is the initial loss, which was set to the same value as in the other two elements. Figure 5-3 (a) shows the simulation results when using a roof element to simulate runoff from the catchment. The response of the roof element is too quick, resulting in an increased peak flow. Figure 5-3 (b) shows the simulation result from the road element, which is similar to the roof element. Both elements produce too much flow volume and catchment response is too quick. Hence, the roof and road element were not suitable for simulating runoff at Broxden.

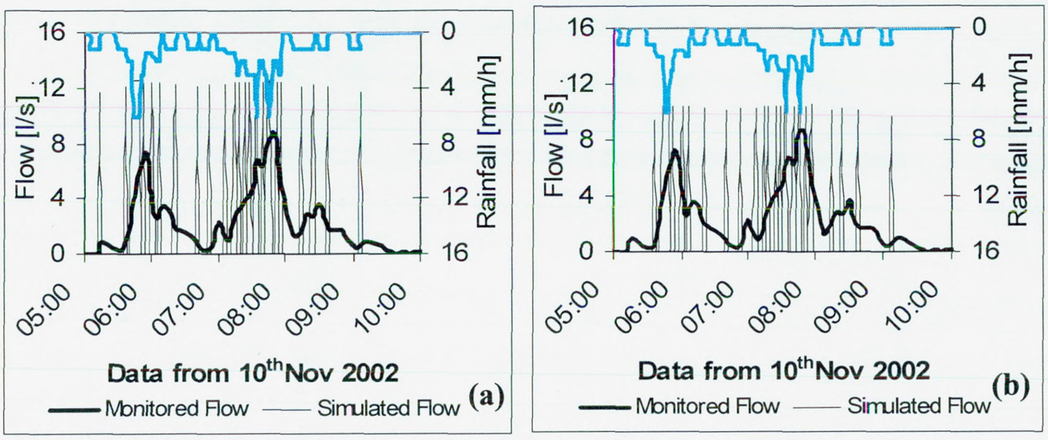



**Figure 5-3: Comparison of runoff modelling elements in Erwin, (a) Roof Element, (b) Road Element**

Figure 5-4 shows simulation results when using the catchment area element. The same parameters as specified in the road element were used but the retention time was varied to fit the monitored flow. Three different retention times were used to demonstrate its influence on the hydrograph. Keeping k (the detention constant) at 2 minutes and increasing n (the number of the reservoirs) from 1 to 5 shows a better fit of the monitored data, as shown in see Figure 5-4.

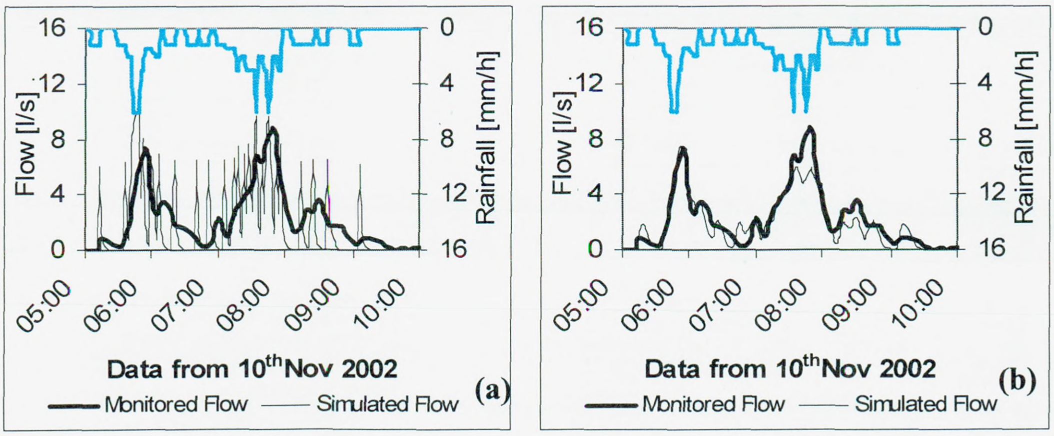



**Figure 5-4: Comparison of catchment area elements in Erwin, (a) 2 min lag time, (b) 10 min lag time**

This is due to the catchments slower response and increase in attenuation. A further increase of n to 10 and a lag time of 20 minutes produces a too slow response, with a too low peak flow.

Comparing the two attenuated hydrographs produced by the catchment area element with the recorded flow shows the attenuated flow behaviour of a cascade of linear reservoirs. This element provides most flexibility for predicting surface runoff and was chosen for simulating surface runoff from the catchment areas.

## 5.7 Sensitivity analyses of surface runoff parameters

Initially five different parameters were available as calibration parameters to predict the surface runoff and these are listed in Table 5-4. The sensitivity analysis was undertaken once, prior to developing the basic runoff model in Erwin to assess the system's sensitivity to changing calibration parameters.

The system at Walker Dam was chosen as example and Figure 5-5 shows results from the sensitivity analyses to predict runoff volume. The graph's steepness and its function show the model's sensitivity according to each parameter. Only the final runoff coefficient and the evaporation factor have a significant impact on the predicted runoff volume. The remaining parameters may have an influence for predicting events separately but were not significant overall.

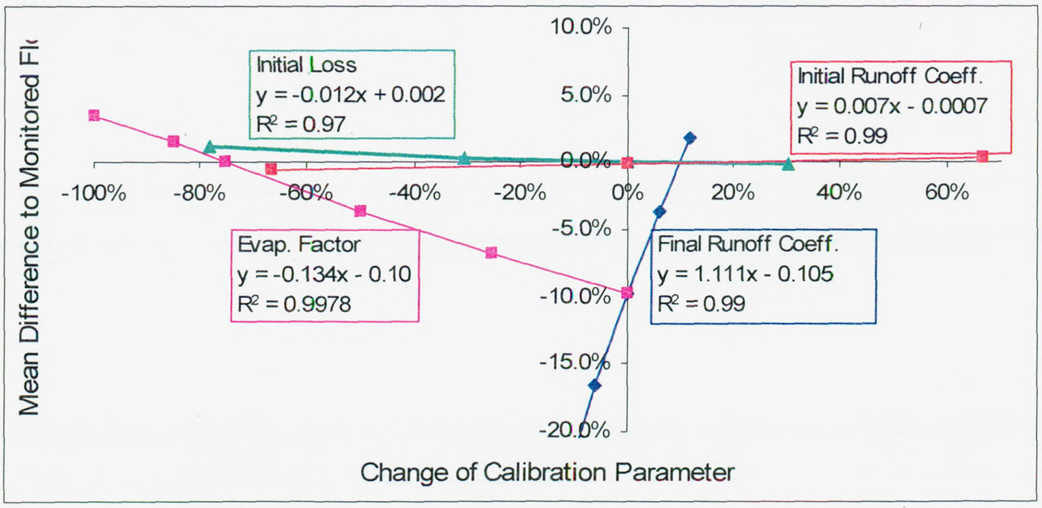



**Figure 5-5: Sensitivity analyses for predicting event volume**

Figure 5-6 shows results from the sensitivity analyses to predict peak flow runoff at Walker Dam, Aberdeen. Only the final runoff coefficient and the linear storage cascade factors, n&k have a significant impact on the predicted peak flow runoff. Table 5-4 shows results from sensitivity analyses (values in bold are default).

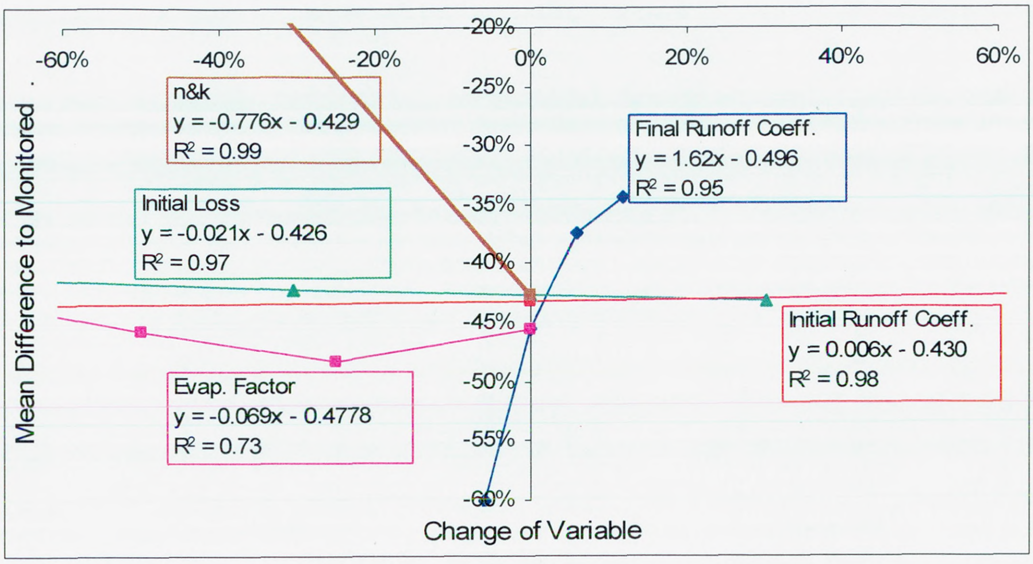



**Figure 5-6: Sensitivity analyses for predicting peak runoff at Walker Dam**

| Calibration Parameter | Value | Change of Variable | Mean Vol. Difference | Mean Abs Difference | Mean Peak Flow Difference |
|---|---|---|---|---|---|
| Percentage Runoff | 75% | -12% | -24.4% | 38.4% | -70.7% |
| | 80% | -6% | -16.6% | 32.8% | -60.0% |
| | **85%** | 0% | -9.7% | 29.6% | -45.5% |
| | 90% | 6% | -3.6% | 27.5% | -37.4% |
| | 95% | 12% | 1.8% | 26.1% | -34.4% |
| Evaporation Factor | 0.00 | -100% | 3.5% | 25.4% | -40.0% |
| | 0.15 | -85% | 1.5% | 25.9% | -41.5% |
| | **0.25** | -75% | 0.1% | 26.2% | -42.7% |
| | 0.50 | -50% | -3.7% | 27.8% | -45.7% |
| | 0.75 | -25% | -6.7% | 28.8% | -48.2% |
| | 1.00 | 0% | -9.7% | 29.6% | -45.5% |
| Initial Loss + Depression Storage | 0.50 | -78% | 1.2% | 26.3% | -40.9% |
| | 1.60 | -30% | 0.4% | 26.2% | -42.1% |
| | **2.30** | 0% | 0.1% | 26.2% | -42.7% |
| | 3.00 | 30% | -0.1% | 26.3% | -43.1% |
| Initital Runoff | 10% | -67% | -0.5% | 26.2% | -43.4% |
| | **30%** | 0% | -0.1% | 26.3% | -43.1% |
| | 50% | 67% | 0.4% | 26.3% | -42.6% |
| n * k | 12 | 0% | 0.4% | 26.3% | -42.6% |
| | **6** | -50% | 0.4% | 26.3% | -5.2% |
| | 4 | -67% | -0.1% | 26.5% | 9.7% |

**Table 5-4: Values from sensitivity analyses of surface runoff**

The evaporation factor was set to the default value of 1 for all systems, as it is thought that Erwin's evaporation loss model represents climatic conditions of Scotland.

The most important parameters to simulate surface runoff were:

- final runoff coefficient
- linear storage cascade parameters n & k.

The remaining parameters were used to fine-tune the model once the main calibration parameters were set.

## 5.8 Hydraulic model testing

The Stormwater Software Package Erwin was used to develop a computer model to simulate outflow from the filter drain. The model was kept as simple as possible to limit the number of calibration parameters. The main part of the hydraulic model consists of a catchment area element, which is connected to a trench soakaway element, as shown in Figure 5-1.

### 5.8.1 Model calibration

The procedure to calibrate each model involved undertaking several runs with varying parameters to try and match the outflow hydrograph using an eyeball comparison. Default values as described in the Erwin User Manual (Ingenieurgesellschaft fuer Stadthydrology, 1997) were used as initial set-up and were then modified to achieve the best fit. To obtain a numerical comparison between recorded and simulated volume, a methodology showing their average difference was also adopted (see Equation (5-1)). The equation incorporates the absolute value of the difference, ensuring positive and negative values do not cancel each other out (Nix, 1994), (Maksimovic and Radojkovic, 1986).

$$F = \frac{\sum |r_i|}{n} \tag{5-1}$$

F [%] is the goodness of fit criterion, $r_i$ [%] is the difference between recorded and simulated volume per event and n is the number of events. The list of parameters that were available to calibrate the models is shown in Table 5-5.

Functions which described the comparison of monitored and simulated flow volume were produced for each site. Details of the modelling results and graphs which were produced to develop this function are shown in Appendix G.

In addition to the adopted volume calibration and eyeball fit, a peak-flow calibration could have been carried out and trials were undertaken at the start of this project. The trials showed that the database package Hydrol, which was used for data analyses did not produce reliable results and the only available method for peak-flow calibration would have been via spreadsheet calculations. This method of data analysis would have been very time consuming and would not have added significant improvement to the models' accuracy.

The system at Transy Estate could not be represented using a generic model as it is influenced by ground water ingress, which Erwin cannot represent.

Table 5-5 provides the calibration parameters which provided the closest fit to the monitored outflow from each model and their meaning is described in section 5.4. The final runoff coefficient and the infiltration rates had the most influence on system flow behaviour. Apart from the amount of depression storage, all values were within the range of standard values as described in section 5.4 and Table 5-1. The depression storages were found to be smaller than the default value and this may reflect the characteristics of the drained areas. All values presented in Table 5-5 were derived using the default parameters as initial set-up and then modified for each new run according to the F-value and the eyeball comparison of the hydrographs.

All systems, apart from the system at Broxden, were located in soil of low permeability and this can be seen by the difference in infiltration rates. This was also the main reason for the difference in performance. The lower infiltration rate of the base at Broxden could be an indication of particle accumulation at the base, which would account for the reduced permeability.

| Location | Initial Loss [mm] | Depression Storage [mm] | Initital Runoff [%] | Final Runoff [%] | Porosity of Fill Material [%] | Infiltration Rate of Base [m/s] | Sides [m/s] |
|---|---|---|---|---|---|---|---|
| Lang Stracht | 0.50 | 0.90 | 20% | 80% | 30% | 0 | $2.0\ 10^{-10}$ |
| Spine Rd | 0.40 | 0.40 | 30% | 95% | 30% | $1.0\ 10^{-6}$ | $2.0\ 10^{-6}$ |
| Glencarse | 0.40 | 0.40 | 25% | 90% | 30% | $1.0\ 10^{-6}$ | $2.0\ 10^{-6}$ |
| Broxden | 0.11 | 0.39 | 30% | 85% | 30% | $7.5\ 10^{-5}$ | $1.0\ 10^{-3}$ |
| Walker Dam | 0.50 | 0.60 | 25% | 80% | 30% | $5.0\ 10^{-8}$ | $2.5\ 10^{-6}$ |

**Table 5-5: Best-fit values of calibration parameters for Erwin models**

In addition to the calibration parameters, the head-discharge relationship was input for each system individually. See Appendix B for details on the head-discharge relationship and section 3.6.4 for monitoring results.

Figure 5-7 shows a comparison of the model accuracy using the function that was developed at each site and Functions which show the model-fit are provided in Table 5-6 with an $R^2$ and an F-value (see Equation (5-1)).

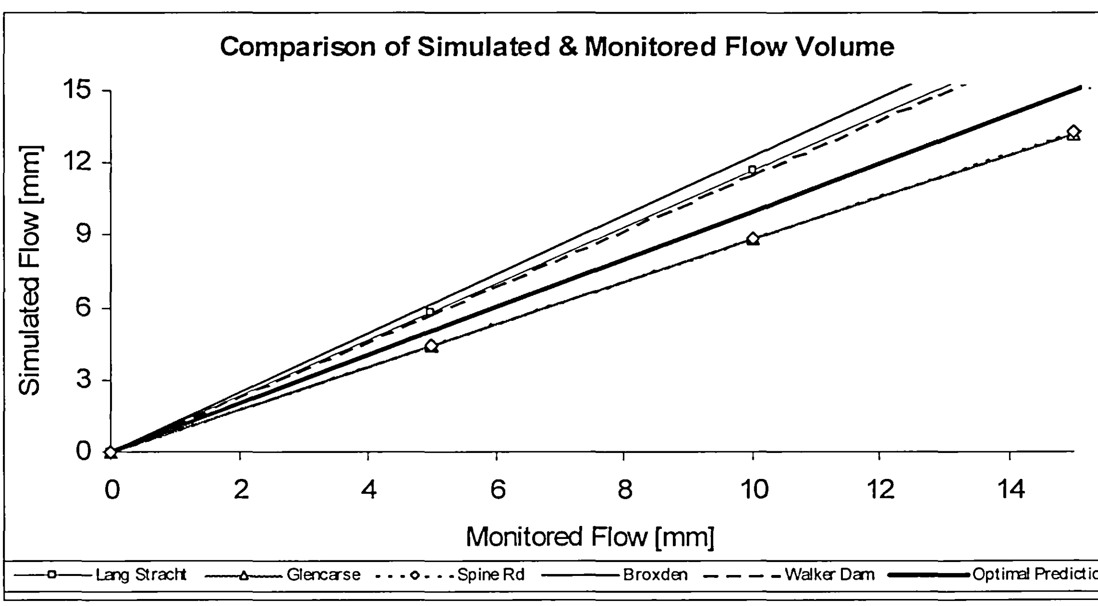



**Figure 5-7: Comparison of model predicting flow volume**

| Location | F(x) Sim_Mon | $R^2$ | F-Value | over / under predicting |
|---|---|---|---|---|
| Lang Stracht | y = 1.11645x | 0.94 | 31% | over |
| Spine Rd | y = 0.8866x | 0.98 | 25% | under |
| Glencarse | y = 0.88528x | 0.89 | 22% | under |
| Broxden | y = 1.2231x | 0.83 | 28% | over |
| Walker Dam | y = 1.1464x | 0.93 | 22% | over |

**Table 5-6: Formulae to predict the outflow**

Details of the modelling results and graphs which were produced to develop these functions are shown in Appendix C and Appendix G.

The models for Walker Dam, Broxden and Lang Stracht were found to over-predict the actual monitoring data slightly, whereas the systems at Glencarse and Spine Road were found to under-predict slightly. F-values range from 22% - 31% and although this is not a perfect fit it shows that each model could be represented using the Erwin package.

### 5.8.2 Performance comparison

Two artificial sets of rainfall data were used to provide a direct performance comparison of the different system designs. The first data set was a time series that contained an extreme rainfall event and this was chosen to show the performance under 'real' conditions. The other data set was a typical 10-year design rainfall event to show the performance under design conditions. Table 5-7 shows the characteristics used to generate the design rainfall.

| M5_60 [mm] | r-value [%] | Return Period [a] | Duration [min] |
|---|---|---|---|
| 16 | 30 | 10 | 30 |

**Table 5-7: Characteristics of design rainfall, M10_30**

Figure 5-8 shows results from the performance comparison, displaying the change in water level at each system, which is taken as level from the invert of the lower drainpipe.

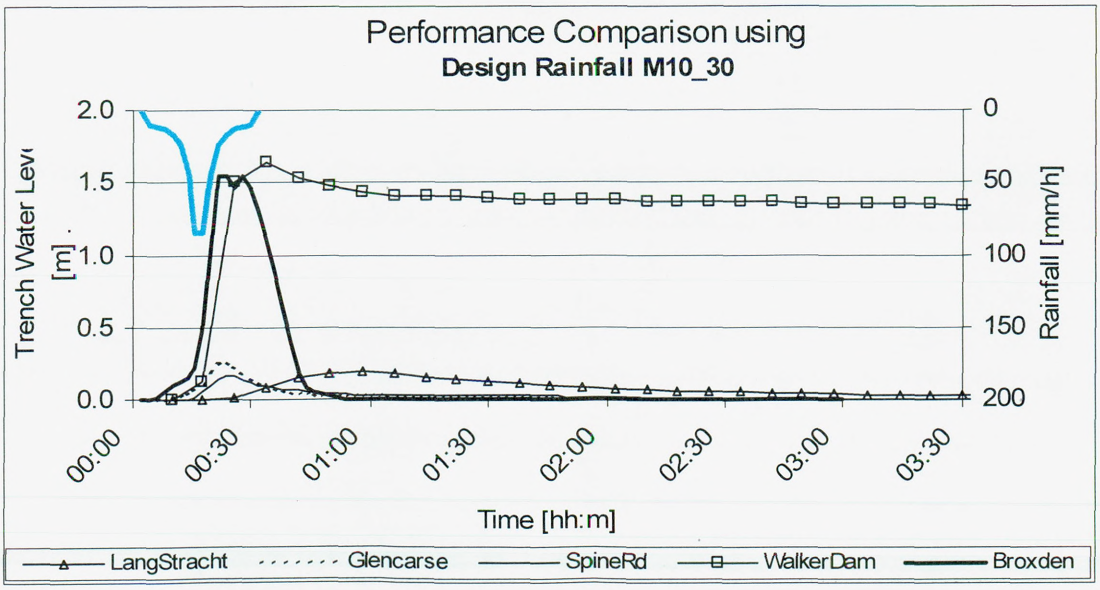



**Figure 5-8: Performance comparison using design rainfall**

It can be seen that the water levels at Walker Dam and Broxden rise over 1.5 m in contrast to the filter drain systems, where the water levels remains below 0.3 m. This comparison shows that all three filter drain systems utilise only approximately 30% of their storage volume, whereas the infiltration trench systems utilise all their storage volume.

The system at Walker Dam remains surcharged for a long period and system emptying does not occur for another 14 days. The reason for this long emptying time is the high level outflow pipe in combination with low permeable soil (see Appendix B.2).

In addition to using design rainfall, a rainfall time series (TS), which was recorded in Edinburgh from April until August 2000 was used to test the systems under 'real' conditions. This rainfall series was chosen, as it contains extreme rainfall events of up to 100 mm rainfall depth per event. Table 5-8 shows simulation results from the design rainfall and the time series data in comparison with the treatment volume. The design treatment volume ($V_T$) according to CIRIA (2000a) and the definition is shown in Equation (5-2).

$$V_T = 9 \bullet D \bullet \left( \frac{SOIL}{2} + \left( 1 - \frac{SOIL}{2} \right) \right) \bullet I \qquad \textbf{(5-2)}$$

D is the design rainfall, I is the impervious fraction of the catchment and SOIL relates to the soil type of the catchment.

| Location | Actual Treatment Volume ($V_T$)* [m$^3$] | Design Treatment Volume ($V_T$)** [m$^3$] | Mean Outflow from TS Rain [%] | Mean Outflow from M10_30 [%] |
|---|---|---|---|---|
| Lang Stracht | 112.5 | 136.8 | 55 | 77 |
| Spine Rd | 45.9 | 44.0 | 57 | 88 |
| Glencarse | 45.0 | 37.8 | 50 | 81 |
| Broxden | 15.2 | 70.5 | 21 | 38 |
| Walker Dam | 8.9 | 62.0 | 50 | 90 |
| Transy Estate | 126.0 | 55.0 | - | - |

* estimated using dimensions and 30% porosity of fill material

** according to standard design (CIRIA, 2000a)

**Table 5-8: Outflow volume in comparison with storage volume**

Results show that the design rainfall as well as the recorded rainfall data produced comparable results. It can be seen that the systems with recorded rain discharged 57 to 21% of rainfall in comparison with 90 to 38% discharge for the design storm. This is mainly due to the rainfall distribution of time series data, which allows better flow reduction through infiltration. Also, there is an increased influence of losses due to evaporation for time series rainfall, which has no influence on design rainfall.

When comparing the system storage volumes and the discharges it can be seen that there is no obvious relationship (see Table 5-8).

However, when comparing the systems' performances whilst using the same soil condition and design rainfall, a strong relationship between the average percentage discharge and treatment volume was found and this is shown in Figure 5-9 (a). Figure 5-9 (b) shows the same comparison when using the recorded rainfall series. A similar trend was found but no relationship could be derived for this scenario. The reason for these findings is discussed in more detail in section 5.9

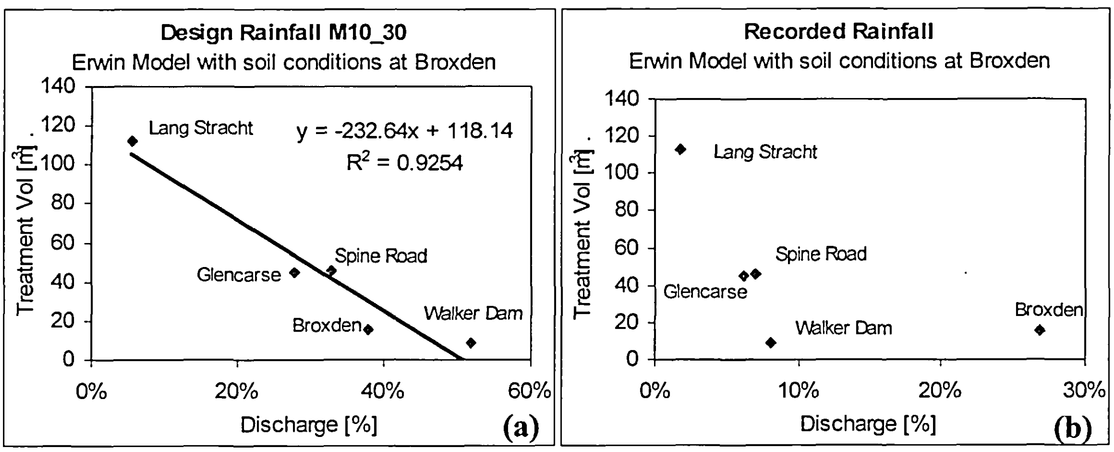



**Figure 5-9: Comparison of treatment volume and discharge, (a) design rainfall, (b) four months recorded rainfall**

## 5.8.3 Influence of soil permeability

To assess the influence of the soil permeability, numerous simulations were carried out with varying infiltration rates. The system at Walker Dam was chosen for this assessment as its performance is most influenced by the soil permeability due to its design with a high level outlet. Simulation runs were undertaken with two scenarios:

- Infiltration rate of the base equal to infiltration rate of the sides
- Infiltration rate of the base equals zero

Setting the infiltration rate of the base to zero represents blockage of the base, which may be due to sediment and debris accumulation. A high groundwater table up to the system's base could also result in zero infiltration through the base due to the reduced hydraulic head. Each scenario was tested using time series and design rainfall (see Section 5.8.2). Figure 5-10 shows the comparison of the percentage outflow and the infiltration rate.

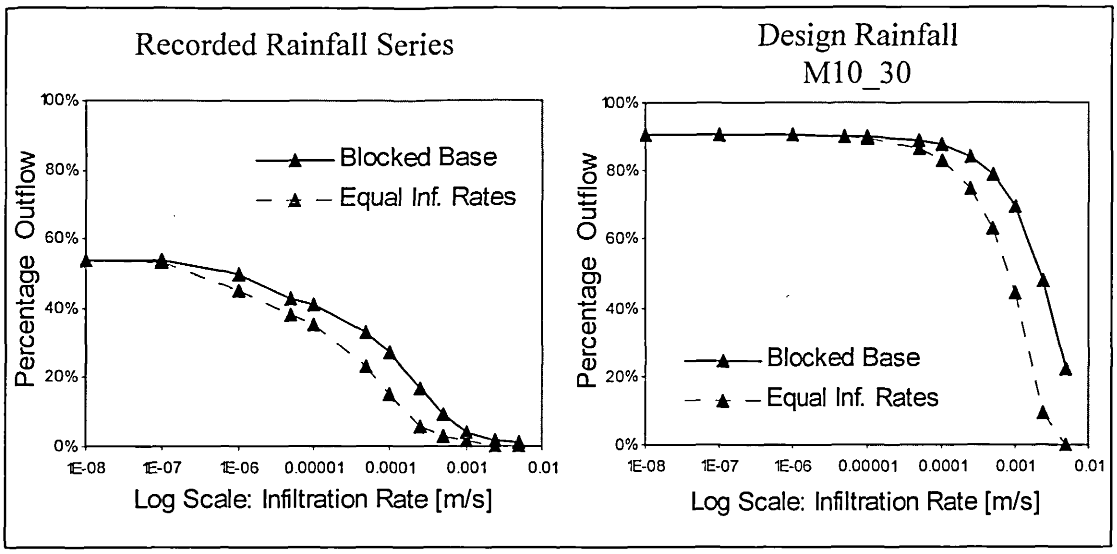



**Figure 5-10: Comparison of percentage outflow with infiltration rate**

Comparison of simulation runs with recorded and generated rainfall shows expected results. Simulation with design rainfall, results in a higher percentage outflow than using the recorded rainfall and this is in line with results presented in section 5.8.2 and Table 5-8. Simulations with recorded rainfall and infiltration rates of less than $1.0*10^{-5}ms^{-1}$ show little difference between the systems with blocked base and systems with infiltrating base. Improved performance with up to 12% flow reduction was found for good drainage conditions with infiltration rates between $2.5*10^{-4}ms^{-1}$ and $5.0*10^{-5}ms^{-1}$ for systems with infiltrating base.

Simulation with design rainfall shows that good drainage conditions with infiltration rates of more than $5.0*10^{-4}ms^{-1}$ is needed to significantly reduce the flow.

## 5.9 Discussion

Flow simulation was successfully undertaken for five of the six systems monitored. The system at Transy could not be simulated successfully as it is influenced by frequent groundwater ingress, which can not be represented with Erwin.

A sensitivity analysis was carried for each system and it was found that systems were most sensitive to the final runoff coefficient and the linear storage cascade parameters n & k. The remaining parameters were used to fine-tune the model.

A good fit was found for all systems simulated with an F-value between 31% and 22% and an $R^2$ between 0.82 and 0.94 (see Table 5-6 and Figure 5-7). Typical hydrographs of simulated and observed outflow are attached for each model in Appendix G. All systems apart from Lang Stracht were simulated successfully for the whole monitoring periods. The system at Lang Stracht could only be represented for a limited period of six months. This was probably due to the site specific problems of blocked gully inlets, which may change their flow capacity over time or additional time varying influences, which the model can not represent.

The treatment volume (VT) of both simulated infiltration trench systems was under-designed by 75 to 80% according to standard design (CIRIA, 2000) (see Table 5-8). Monitoring and modelling results showed that the system at Broxden provided the highest flow volume reduction of all systems. The main reason for the improved performance was due to good drainage-conditions of the soil with infiltration rates of $1.0*10^{-3}ms^{-1}$ (Whitlow, 2001). The enhanced reduction in flow was supported by the use of small diameter drainpipes which allowed system surcharge, promoting exfiltration (see Appendix G.4). The system emptying time was less than one hour using a 10-year design storm and no overflow occurred (see Figure 5-9). Monitoring and modelling of the infiltration trench at Walker Dam gave evidence that the system is located in low permeable soil and system emptying lasted two weeks after the design storm. Monitoring showed a satisfactory performance despite the long emptying time, i.e. no flooding occurred and outflow was reduced by 34%. Similar results were published by Abbott and Camino-Mateos (2001), who reported on the satisfactory performance of a soakaway, despite emptying times of up to two weeks.

The calculation of VT for the three filter drain systems provided a good estimate of the actual storage volume provided (see Table 5-3). However, modelling results from design storm and the recorded rainfall showed that the water level never increases by more than 0.3 m (above the pipe invert) showing that the remaining storage volume was never activated. All filter drain systems were located in soils of low permeability and hydraulic simulation showed little reduction of flow volume through infiltration.

When comparing the systems' on-site performances, no significant influence of the storage volume was found. The filter drain systems discharge most of the flow volume, which is due to the low elevation of the drainpipe and the poor permeability of the soil.

However, when comparing the hydraulic behaviour of all systems in soil conditions as found at Broxden, a strong relationship between flow discharge and treatment volume was evident when using design rainfall (see Figure 5-9 (a)). A similar trend was found for recorded rainfall but no relationship could be derived (see Figure 5-9 (b)). This is due to the difference in detailing, which has an increased influence for small to medium events but not for design rainfall; i.e. flow reduction at Walker Dam improves by almost 50%, when comparing outflow from recorded with that from design rainfall (see Figure 5-9), whereas flow reduction at Broxden improves by only 10%. The main difference between the two models is the elevated outlet at Walker Dam and this is thought to be the reason for the improved outflow reduction during small to medium events. The system at Broxden performs better than Walker Dam when using design rainfall because of its larger treatment volume and worse when using recorded rainfall due to the low elevation of the system outlet. This scenario shows the importance of a high level outflow in combination with good permeable soil, which results in better flow reduction while using the relatively small storage volume at Walker Dam.

Soils with high infiltration rates are required to provide significant reduction in percentage outflow when using design rainfall (see Figure 5-10). This is due to the extremely high flow rate produced by the design rainfall and shows that the system at Walker Dam would have little impact during extreme events. Better flow attenuation for extreme events may be expected for systems with larger storage volume.

Using recorded rainfall shows that flow volume reduction of more than 50% was achieved in very poor drainage conditions with infiltration rates below $1.0*10^{-7}ms^{-1}$ and this is mainly due to surface runoff losses. Surface runoff losses were found to have a significant influence on small to medium events but limited influence for extreme events or design rainfall.

The Erwin models enabled a performance comparison between the different sites using design rainfall and time series rain data, including extreme rain events, which showed that the soil permeability has a major influence on the system's flow attenuation.

However, extensive flow monitoring had to be undertaken to successfully represent the systems' hydraulic behaviour. One of the most important input data was the head-discharge relationship of the outlet pipe. This relationship had to be extended to represent flow within the gravel and these values were input to fit the model. To date no standard program is available to represent flow through gravel material without the need of extensive monitoring and the use of head-discharge relationships.

The next chapter provides details of the FVD model, which enables simulation of flow through gravel filled trenches without the need of extensive flow monitoring.

# CHAPTER 6 DEVELOPMENT OF THE FINITE-VOLUME DARCY'S FLOW (FVD) MODEL

*This chapter provides information about the newly developed Finite-Volume and Darcy's Flow (FVD) model, which was commissioned by HR Wallingford. The FVD model enables the user to represent flow through gravel filled trenches without the need of extensive flow monitoring data. Initially Darcy's Law was combined with the flow volume approximation to represent flow through gravel. This model was built using Excel and then translated to a script-based model. This enabled model validation using monitored data from the infiltration trench systems at Walker Dam.*

## 6.1 Introduction

A review of available literature and discussion with key specialists from leading software companies in the field of urban drainage showed that to date no standardised programs were available to represent flow through in-ground SUDS.

Software packages, such as Erwin and WinDes (see Section 2.5), which provide flow simulation through in-ground SUDS use predefined or user specific relationships to represent the flow characteristics of infiltration trenches and filter drains. This approach has the disadvantage that substantial monitoring data has to be available for calibration and verification to enable a satisfactory representation of such systems. In many cases this information is not available due to time and financial constrains, and extrapolation of such black-box models to extreme conditions is not possible. An alternative approach based on physical characteristics of the system was desirable.

The following section provides detailed information on the Finite Volume Darcy's Flow (FVD) model, which is presented here for the first time. The model development was undertaken in collaboration with HR Wallingford, one of the leading hydraulics research organisations in the field of urban drainage. The integration of the FVD model into HR Wallingford's popular Infoworks package is proposed for the next release (Version 6.5).

An overview of Darcy's Law and finite volume method is followed by a section on the development of a model of a fictitious trench system. The initial model was developed using a spreadsheet, which was then translated to a code-based procedure.

## 6.2 Rationale

Today's standardised computer models for simulating flow through in-ground SUDS require substantial monitoring data in order to allow for a realistic representation of system behaviour, as they are based on predefined or user-specific head-discharge-relationships.

Darcy's law in combination with the finite volume method was applied to compute the flow through filter material and validated against monitored data. The FVD model is based on physical characteristics, which represents flow through gravel material. Flow characteristics are dependent on the dimensions and materials used.

## 6.3 Methodology

The initial model development was undertaken using a spreadsheet, which offered efficient control over input and output data. Simulation results were instantly visible in numerical and graphical form.

Only a limited amount of data could be computed due to mathematical constraints and the spreadsheet had to be translated into a code-based model. The computer package Hydrol Time Series Model was used for the computation and Hydrol Time Series Manager was set-up as database for model input and output. This arrangement offered maximum flexibility in terms of time interval and simulation period.

Initially, a sensitivity analysis was undertaken using a fictitious trench system allowing the testing of various parameters. The model validation was undertaken using recorded data from the study site at Walker Dam.

## 6.4 Overview of Darcy's law and finite volume methods

Using a series of sand column experiments Henry Darcy established in the 1850s that, for a given type of sand, the volume of discharge rate Q ($m^3\ s^{-1}$) is proportional to the head difference $\Delta H$ and to the cross-sectional area A ($m^2$) of the column. Discharge Q can be calculated using Darcy's law from Equation (6-1), K ($m\ s^{-1}$) being the hydraulic conductivity of the soil.

$$Q = -KA\frac{\Delta H}{\Delta L} \tag{6-1}$$

Equation (6-1) can be rewritten in a general form:

$$Q = -KA\frac{\triangle H}{\triangle x} \qquad \textbf{(6-2)}$$

and then combined with the following volumetric mass- balance:

$$\mu\frac{\triangle H}{\triangle t} = -\frac{\triangle Q}{\triangle x} \qquad \textbf{(6-3)}$$

to model flow through in-ground SUDS, where μ is the specific yield and this is generally smaller than porosity because of the capillary fringes.

Several models have been developed, using Darcy's law as their basis to describe the effects on the change of water level due to gravitational flow. These can be classified as follows (Smart and Herbertson, 1992):

- Analytical models
- Finite difference models
- Finite volume models

Analytical models use Darcy's law and the continuity equation to determine relationships, which are used to calculate the change of water levels. Simple systems can be analysed without the aid of computers but computation of more complex systems is inevitably connected to the use of a computer.

## 6.5 *Modelling packages used for running the model*

The initial simulation, as described in section 6.6.1, was undertaken using a spreadsheet-based model. Spreadsheet models have advantages during model development as they contain a fixed data set which offers maximum control. The flow computation is instantaneous and straightforward and errors are easily spotted using the built in scatter graphs. The main disadvantage of the spreadsheet model is that only a very limited amount of data can be computed and there is little flexibility.

The finite volume method is an integration of the mass balance and Darcy's law and is described in section 6.6.2. It required a code-based model, as the calculations were too complex for the spreadsheet calculation. File sizes also became too large.

A language similar to Visual Basic, named Hydrol Basic, was used to translate the spreadsheet model to a code-based model. Hydrol Time Series Manager and its associated Modelling Package were used to carry out the simulation. The code based model offered greater flexibility for the following main reasons:

1. System inflow is the input to the model and this can be accessed directly from the Hydrol database and computed to the desired time step. Hydrol performs automatic interpolation from recorded data points.
2. The code-based model allows computation of any programmable process using Hydrol Basic.
3. Model output can be set-up in several ways; in numerical form as an ASCII file; direct to the Hydrol database; or in graphical form as an on-screen bitmap file. Graphical output also allows instant comparison with additional data sets, such as recorded flow or rainfall data.

It should be noted that the Hydrol Time Series Program is not essential for running the FVD model, as the code for the FVD model could have been computed using any script-based program. Reference should be made to section 2.5.4 for more information about Hydrol Time Series Manager.

## *6.6 Model development using a fictitious trench system*

Initial model development was undertaken using a fictitious infiltration trench system, which was computed with a spreadsheet model. The fictitious trench system was used to simplify the model development as much as possible.

### 6.6.1 Darcy's law and volumetric balance using a spreadsheet model

The model was used first on the system shown in Figure 6-1. It is a simple infiltration trench system, which has neither perforated drainpipes nor any flow restrictions downstream.

The model behaviour was checked against the change of various parameters. A list of parameters used in the model is shown in Table 6-1.

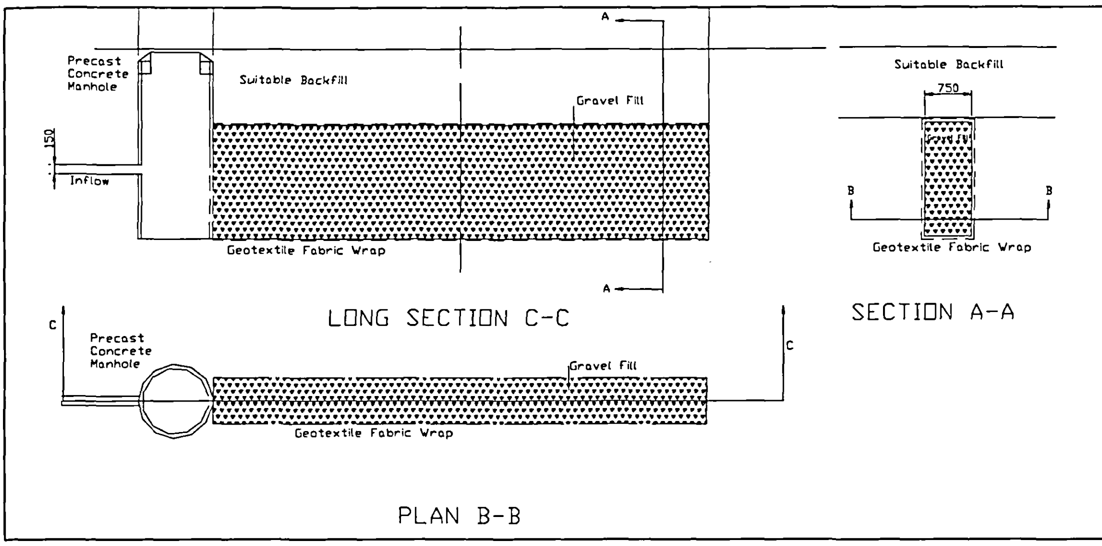



**Figure 6-1: Fictitious model application**

| Parameters [unit] | Example Value |
|---|---|
| Hydraulic conductivity of fill material [ms-1] | 0.01 |
| Length [m] | 50 |
| Width [m] | 0.8 |
| Depth [m] | 1.5 |
| Exfiltration Rate Sides [ms-1] | $1*10^{-6}$ |
| Exfiltration Rate Base [ms-1] | $1*10^{-6}$ |
| Porosity [%] | 30 |

**Table 6-1: Parameters used in fictitious model application**

The precast concrete manhole receives inflow which is distributed into the gravel filled trench from the manhole. The manhole is modified in such a way that it allows inflow into the trench through an opening along the side adjacent to the trench. The calculation of manhole level ($H_{Manhole}$) and manhole volume ($V_{Manhole}$) is determined by a volumetric balance according to Equation (6-4).

$$V_{Manhole_{(t+\Delta t)}} = V_{Manhole_{(t)}} + V_{Inflow_{(t+\Delta t)}} - V_{Outflow_{(t)}} \tag{6-4}$$

$V_{Inflow}$ and $V_{Outflow}$ are the volumetric in- and outflow ($m^3$), respectively. The inflow into the trench is calculated according to Darcy's law using the head difference between the manhole and the trench according to Equation (6-5), where D is the diameter (m) of the manhole.

$$H_{Manhole_{(t)}} = \frac{V_{Manhole_{(t)}}}{\pi \cdot \frac{D^2}{4}} \qquad \textbf{(6-5)}$$

The flow volume ($V_{Trench}$) [$m^3$] is calculated using volumetric balance as shown in Equation (6-6).

$$V_{Trench_{(t+\Delta t)}} = V_{Trench_{(t)}} + V_{Inflow_{(t+\Delta t)}} - V_{Outflow_{(t)}} - V_{Exfiltration_{(t)}} \qquad \textbf{(6-6)}$$

The water level within the trench ($H_{Trench}$) is calculated according to Equation (6-7), where the characteristics of the trench are width (w), length (L) and porosity of fill material (P).

$$H_{Trench_{(t)}} = \frac{V_{Trench_{(t)}}}{w \cdot L \cdot P} \qquad \textbf{(6-7)}$$

Losses due to exfiltration ($V_{Exfiltration}$) are taken into account from the trench sides and base as shown in Equation (6-8) and (6-9).

$$V_{Exfiltration_{(t)}} = Q_{TrenchLosses_{(t)}} \cdot \Delta t \qquad \textbf{(6-8)}$$

$$Q_{TrenchLosses} = Q_{LossesSides} + Q_{LossesBase} \qquad \textbf{(6-9)}$$

Exfiltration losses from the side, $Q_{LossesSides}$ are calculated according to Equation (6-10), where $K_{Sides}$ [ms-1] is the hydraulic conductivity of the side-wall of the trench, I is the slope of water flow (see **Error! Reference source not found.** under Assumptions on page **Error! Bookmark not defined.**), $Area_{Sides}$ is the cross sectional area of the trench sides and $Q_{LossesBase}$ as shown in Equation (6-12).

$$Q_{LossesSides} = K_{Side} \cdot I \cdot Area_{Sides} \qquad \textbf{(6-10)}$$

$$Area_{Sides} = H \cdot L \qquad \textbf{(6-11)}$$

Losses from the base, $Q_{Base}$ are calculated in the same way as from the sides and the relevant formulae are given in Equation (6-12) and (6-13) for completeness.

$$Q_{LossesBase} = K_{Base} \cdot I \cdot Area_{Base} \tag{6-12}$$

$$Area_{Base} = w \cdot L \tag{6-13}$$

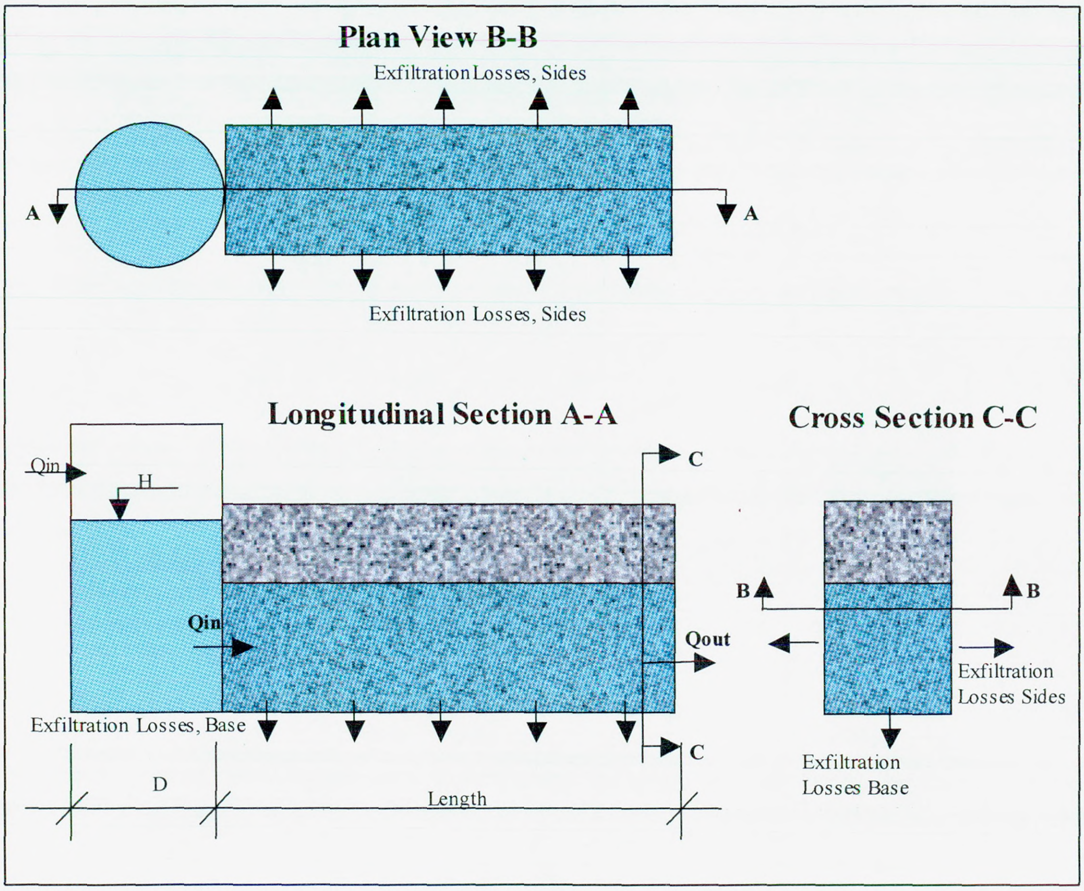



**Figure 6-2: Flow paths using Darcy law only**

The disadvantage of this model is its limited accuracy, which is a result of calculating the flow volume for the gravel system as one unit. The model may provide sufficient accuracy for small systems but unsatisfactory results are expected when representing medium to large systems.

To improve model accuracy the finite volume approximation is introduced in combination with Darcy's law in the following section.

### 6.6.2 FVD calculation used in a code model

The system is divided into (n) number of cells as shown in Figure 6-3 and Darcy's law is used to calculate the flow from cell (i) to cell (i+1) using linear relationships as shown in Equations (6-14) to (6-18).

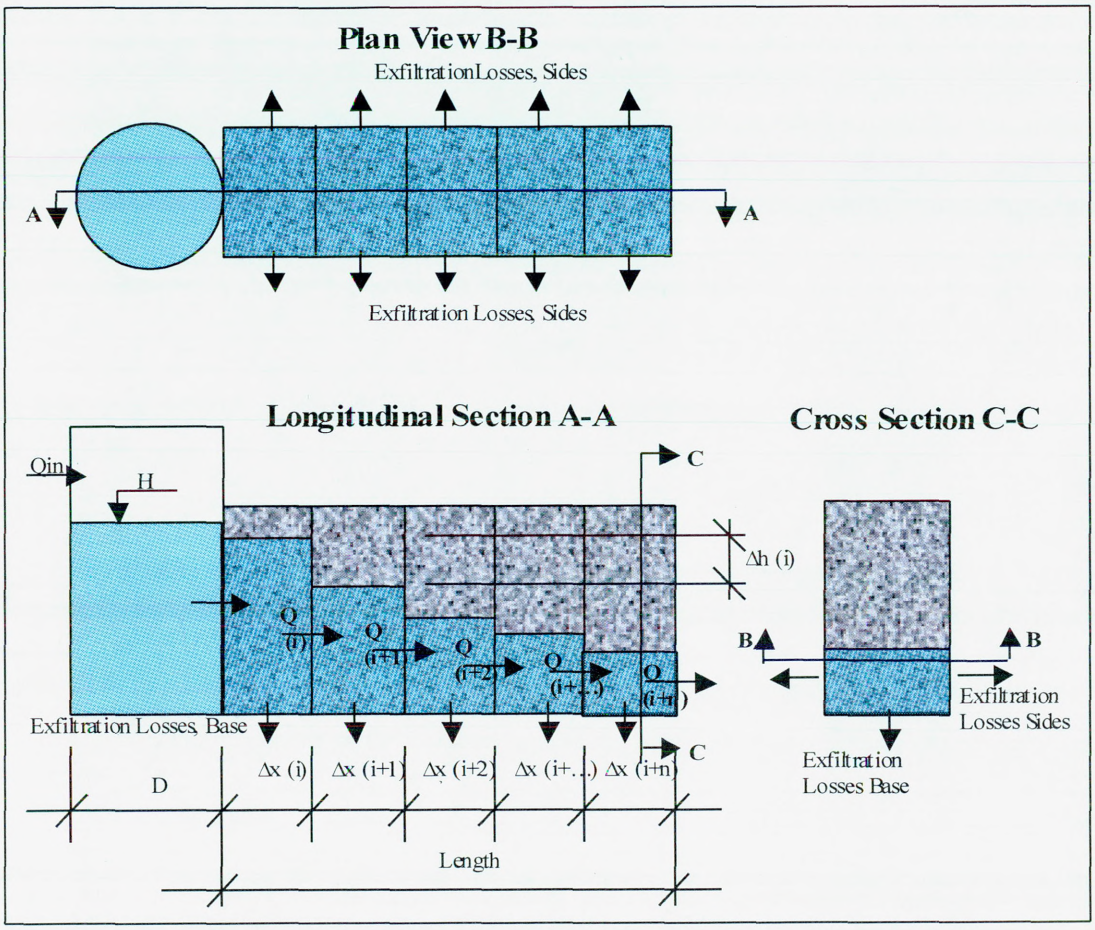



**Figure 6-3: Schematic of flow paths using the FVD model for the fictitious trench system**

$$Q_{cell(i)} = K_h \cdot \widetilde{Area}_{(i+1)} \cdot \left[ \frac{\Delta h_{(i)}}{\Delta x_{(i)}} \right] \quad \textbf{(6-14)}$$

$$\widetilde{Area}_{(i+1)} = \frac{Area_{(i)} + Area_{(i+1)}}{2} \quad \textbf{(6-15)}$$

$$Area_{(i)} = H_{Cell(i)} \cdot w_{Cell(i)} \quad \textbf{(6-16)}$$

$$Area_{(i+1)} = H_{Cell(i+1)} \cdot w_{Cell(i+1)} \quad \textbf{(6-17)}$$

$$\Delta h_{(i)} = H_{Cell(i)} - H_{Cell(i+1)} \quad \textbf{(6-18)}$$

$Q_{cell(i)}$ ($m^3s^{-1}$) is the discharge from $cell_{(i)}$ to $cell_{(i+1)}$, $K_h$ ($ms^{-1}$) is the hydraulic conductivity of fill material, $\widetilde{Area}_{(i+1)}$ ($m^2$) is the average cross section area of cell (i+1) and $\Delta h_{(i)}\Delta x_{(i)}^{-1}$ (dimensionless) is the water surface gradient.

Exfiltration losses, water level and volume are calculated for each cell in the same way as for the whole trench using Equation (6-6) to (6-13). The accuracy of flow prediction increases with the number of cells but this is also limited by Equation (6-19).

$$\frac{K_h \cdot \Delta t}{\Delta x^2 \cdot P} < \frac{1}{2} \quad \textbf{(6-19)}$$

The physical interpretation of the constraint Equation (6-19) is that when the inflow is larger than outflow the net water increase in the cell cannot exceed the cell capacity; when the cell is losing water because outflow is larger than inflow, the net water decrease is not allowed to over-dry the cell. Mathematically, this is the criterion which ensures the FVD model is stable (Huyarkon and Pinder, 1983). Therefore, care has to be taken when choosing the $\Delta t$, $\Delta x$ and $K_h$. Values of $\Delta t$ which are too large, or $\Delta x$ too small or $K_h$ too high will result in system instability. Figure 6-4 shows that the model for the fictitious trench system crashes when selecting a time step of >65 sec. This is because of the cell sizes, which receive more flow volume than could be stored during one time-step. This results in water levels rising infinitely, which causes system instability. Similar characteristics were found when increasing the number of cells to over 60 or decreasing the porosity to below 0.1%.

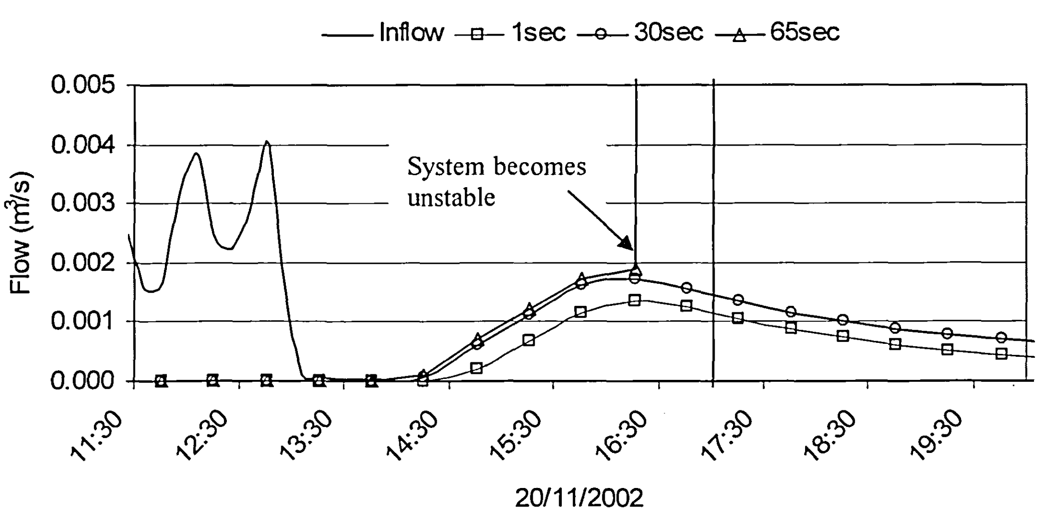



**Figure 6-4: Influence of time-step**

The time step to compute flow was set to one second for simulation runs to enable a high degree of accuracy while applying Equation (6-19). Data were then converted to two-minute intervals and stored in Hydrol TSM (Hydrol Time Series Manager, 2000), which improved data management and reduced file sizes. Figure 6-5 is a flow chart of the model and Figure 6-6 shows the FVD code in Hydrol Basic (Hydrol Modelling, 1999).

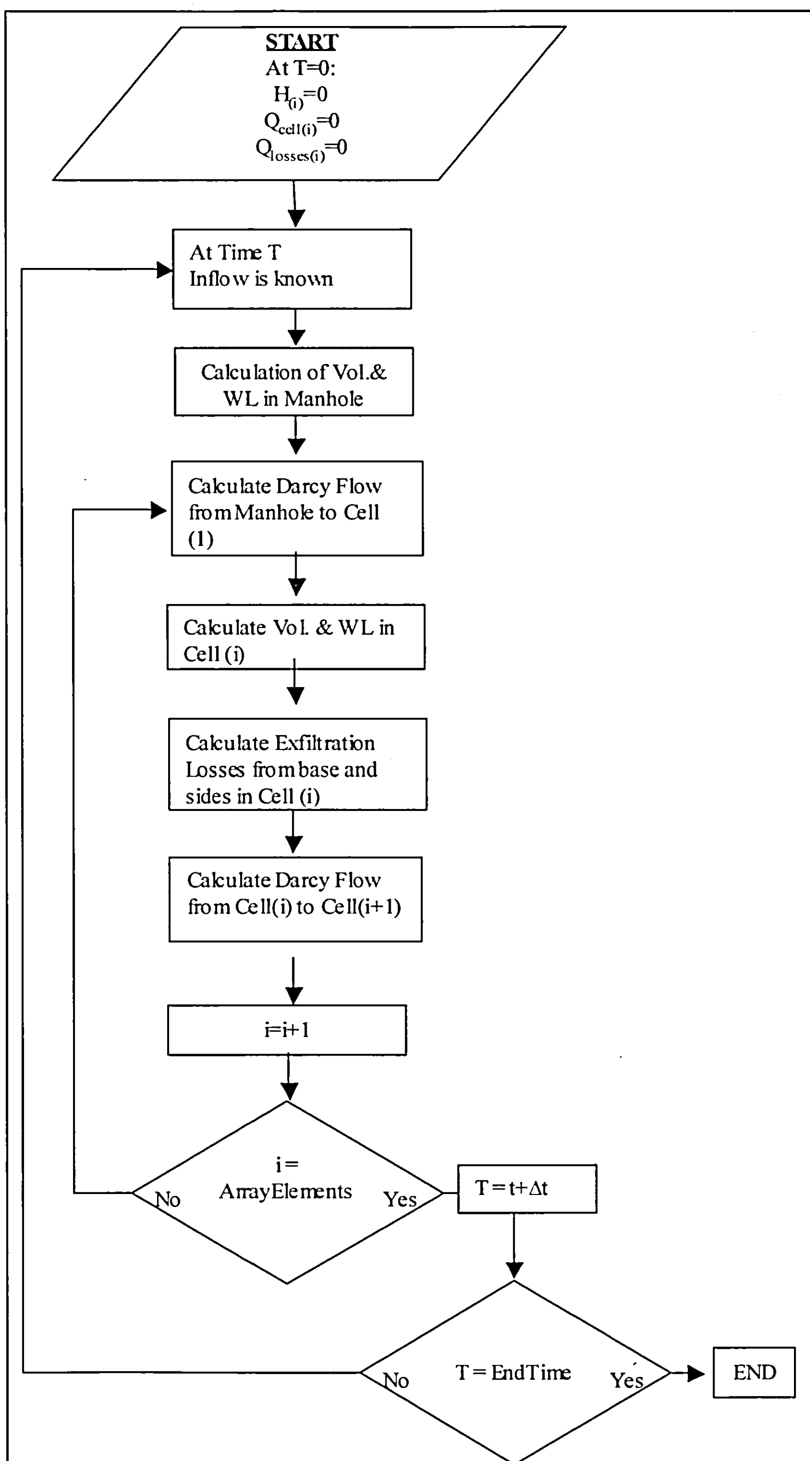



**Figure 6-5: Basic flow chart of FVD Model**

```
;--------------------------------------------------------------------
; VARIABLE DECLARATIONS (with added Definitions)
;--------------------------------------------------------------------
float dx=0                      Cell length [m]
float VM=0                      Volume of Water in Manhole [m3]
float Q[ArrayElements] = 0      Darcy Flow in Cell [m3s-1]
float H[ArrayElements] = 0      Water Level in Cell [m]
float V[ArrayElements] = 0      Volume of Water in Cell [m3]
float Vx[ArrayElements]=0       Volume of Exfiltration [m3]
int i = 0                       Number of Cell, counting
const int ArrayElements=10      Total Number of Cells
float Ab=0                      Area of Manhole Base [m2]
float D=1.2                     Manhole Diameter [m]
Inflow                          Recorded Inflow [lm3s-1]
STEP                            TimeStep [h]
float HM=0                      Water Level in Manhole [m]
const float KH=0.01             Hydraulic Conductivity [ms-1]
const float w=0.8               Trench Width [m]
const float Length=500          Trench Length [m]
const float kv= 1e-005          Infiltration rate of base [ms-1]
const float ks= 1e-005          Infiltration rate of sides [ms-1]
const float P=0.3               Porosity of Fill Material[-]
;------------------------------------------------------------
;ManholeCalculations
;------------------------------------------------------------
        Ab=PI*D^2/4
        Vin=Inflow*STEP*3600
        VM=VM+Vin-Q[1]*STEP*3600
        HM=VM/Ab
;------------------------------------------------------------
;TrenchFlow Calculation
;------------------------------------------------------------
        dx=Length/ArrayElements
        i = 1
        Do While (i<ArrayElements)
                H[i] = V[i]/(w*dx*P)
                Q[1]=Kh*(HM+H[1])*0.5*w*(HM-H[1])/dx
                V[i] = V[i]+Q[i]*STEP*3600-Q[i+1]*STEP*3600-Vx[i]
                        If (V[i]<0) Then
                        V[i]=0
                        End If
                        If (H[i]<=0) Then
                        Vx[i]=0
```

**Figure 6-6: Basic code for FVD model**

### 6.6.3 Assumptions inherent in the FVD Model

(1) Darcy's law was derived from steady state flow, but proven to be valid to model unsteady flow as long as the flow is in laminar condition. If the flow becomes turbulent the modified Darcy's law method to include a non-linear term must be used.

(2) The capillary effect is negligible due to the grain size of fill material which is relatively large, so the specific yield was taken to be the porosity.

(3) Exfiltration rates are constant. Only losses from the bottom and the sides are taken into account; no losses are deducted from the top or ends of the system.

(4) There is no change in hydraulic conductivity over time, no sedimentation nor clogging of the system is taken into account, i.e. exfiltration rates, hydraulic conductivity of fill material and porosity remain constant over time.

(5) A one dimensional calculation is applicable (see below)

### 6.6.4 Number of dimensions for the FVD model

As filter drains and infiltration trenches are longitudinal systems, a one-dimensional approach to calculate the flow was considered to be valid. Flow in the y and z directions could be taken into account but the improvement would not be significant. Most flow through filter drains and infiltration trenches occurs in the horizontal. This is especially true for systems in low permeability soils. Flow in the y and z directions would not be modelled but would be accountable through the volumetric balance of the loss model, which calculates the volumetric change according to the water level and loss rate.

Using a one-dimensional model also saves computational time and keeps file sizes manageable. Large file sizes become problematic, when using very short time-steps.

A two-dimensional approach would be valid to model a pervious pavement or infiltration basin, where water flows both in the x and y-directions.

### 6.6.5 Sensitivity analyses

A sensitivity analysis was carried out for the fictitious trench system, as described in Figure 6-1, using an eyeball comparison of the hydrographs produced by the FVD model as well as a numerical comparison of flow volumes. This sensitivity analysis was undertaken to show the sensitivity of the FVD model to each parameter. An artificial inflow was used and in the absence of monitored outflow the sensitivity analysis was carried out by comparing the modelled outflow hydrographs. Several scenarios were simulated by changing one parameter and leaving the remaining parameters fixed as default values. Figure 6-8 and Figure 6-9 show the inflow and outflow hydrographs produced by the FVD model. Figure 6-10 shows the sensitivity of each parameter. Table 6-2 shows the values of each parameter used in the sensitivity analysis together with the outflow volume from the FVD model. Values in bold are default values.

| Parameter | Value | Change of Value | Outflow Vol [m$^3$] | Change of Volume |
|---|---|---|---|---|
| Number of Cells (n) | 2 | -80% | 0.20 | 102% |
| | 5 | -50% | 0.12 | 21% |
| | **10** | 0% | 0.10 | 0% |
| | 60 | 500% | 0.08 | -16% |
| Hydraulic Conductivity (kh) | 0.1 | 900% | 0.24 | 141% |
| | 0.05 | 400% | 0.21 | 109% |
| | **0.01** | 0% | 0.10 | 0% |
| | 0.002 | -80% | 0.00 | -97% |
| Porosity (P) | 10% | -67% | 0.17 | 68% |
| | 20% | -33% | 0.13 | 31% |
| | **30%** | 0% | 0.10 | 0% |
| | 50% | 67% | 0.05 | -47% |

**Table 6-2: Parameters for FVD model**

The total number of cells (n) influences the accuracy of the computation in the FVD model. The greater the number of cells is, the higher the resolution of flow along the trench and the better the accuracy of the model. Figure 6-7 shows a much quicker response with over 100% flow volume increase when using two cells as compared with 10 cells. The difference between results of flow volume from the run with 60 cells as compared with 10 cells is only -16%.

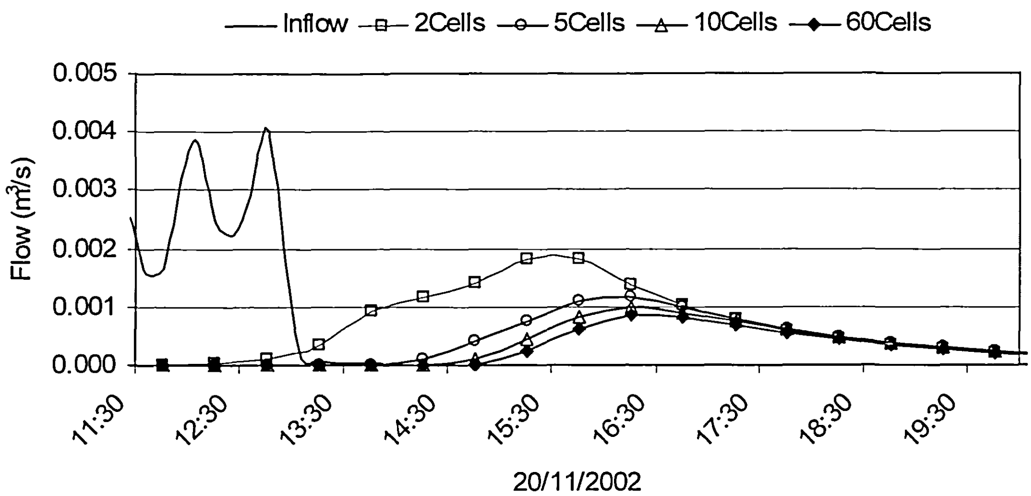



**Figure 6-7: FVD model showing the influence of the number of cells (n)**

The hydraulic conductivity of fill material ($K_h$) has the greatest influence on the attenuation of flow. This can be seen when comparing modelling results of Kh0.1ms$^{-1}$ with the default value of $K_h$ 0.01ms$^{-1}$ (see Table 6-2). The increased hydraulic conductivity produces 141% more flow volume, outflow starts two hours earlier and the peak flow is three times higher. When changing $K_h$ to 0.002ms$^{-1}$ there is no outflow from the system but all flow is retained and lost through exfiltration.

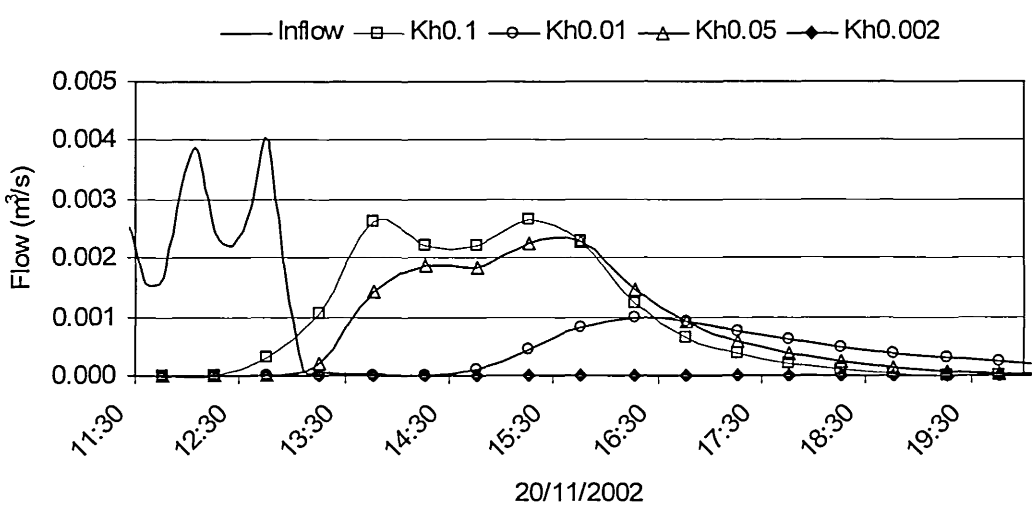



**Figure 6-8: FVD model showing the influence of $K_h$**

The porosity of fill material also plays an important role in flow volume reduction. Fill material with porosity of 50% produces almost 50% less outflow in comparison with fill material of 30% porosity.

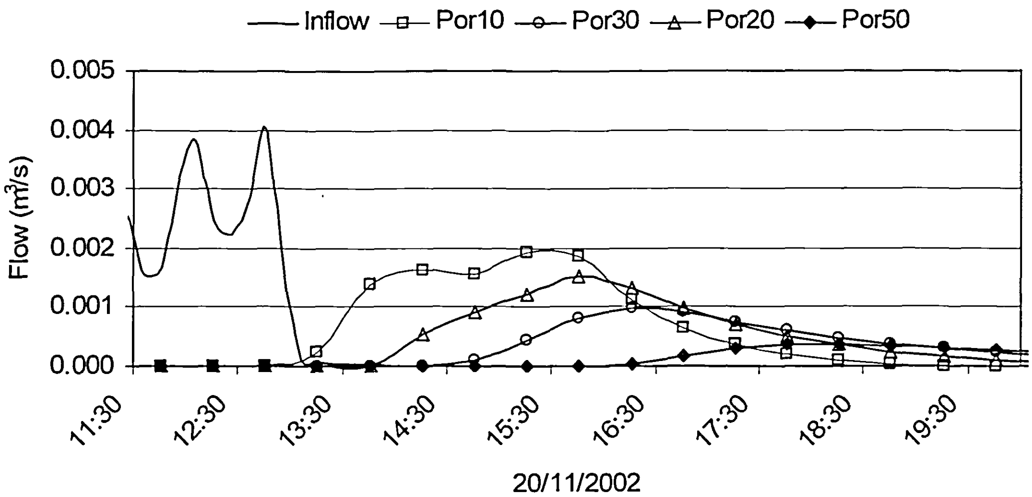



**Figure 6-9: FVD model showing influence of Porosity**

The sensitivity of each parameter can be seen in the steepness of the curves in Figure 6-10. The number cells (n) has a maximum influence on the predicted outflow when using very few cells only and this influence decreases asymptotically with the increase of number of cells. n should be used to maximise accuracy, while allowing short computation times. Physical dimensions restrict the number of cells and this depends on the flow volume and the computational time-step (see Equation (6-19)).

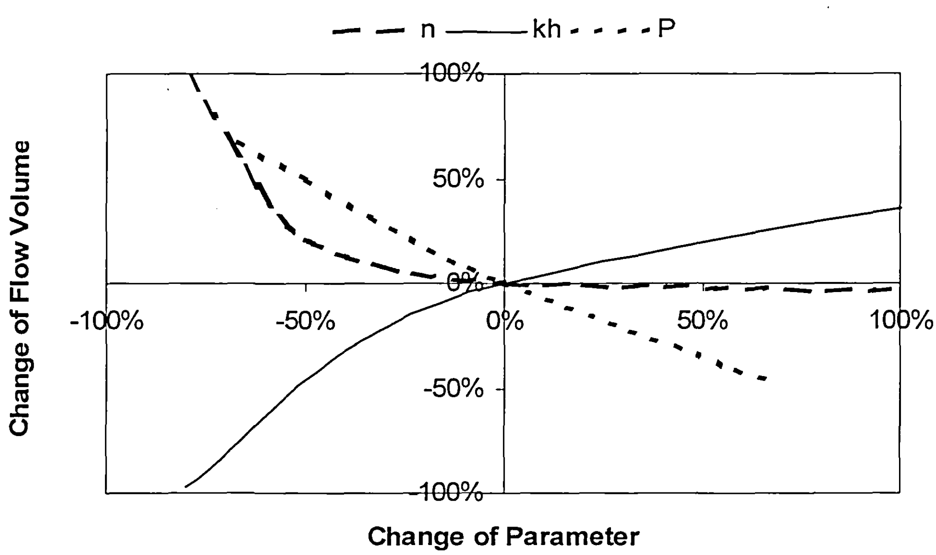



**Figure 6-10: Sensitivity analyses of FVD model**

The porosity (P) and the hydraulic conductivity ($K_h$) are both material-dependent parameters, which have a similar influence on the sensitivity of the model. The value for the hydraulic conductivity can be estimated from on-site measurements or obtained from literature. Values for the porosity of the fill material can also be obtained from the literature but the porosity is expected to reduce over time as the system silts up. This process depends on the system's inflow characteristic and the maintenance carried out (see Section 4.8.)

## 6.7 Model validation using the study site at Walker Dam

### 6.7.1 Site specific data

The infiltration trench at the study site Walker Dam, Aberdeen was chosen for model validation, as it is a relatively small system and its flow characteristics are well understood through monitoring. Inflow into the gravel is through the upper end of the system via a short perforated pipe. The lower perforated pipe can be neglected for calculating flow as the system outlet is raised. The use of a high level outlet maximises the potential for system storage and provides good flow attenuation. A detailed construction drawing is shown in Figure 6-11.

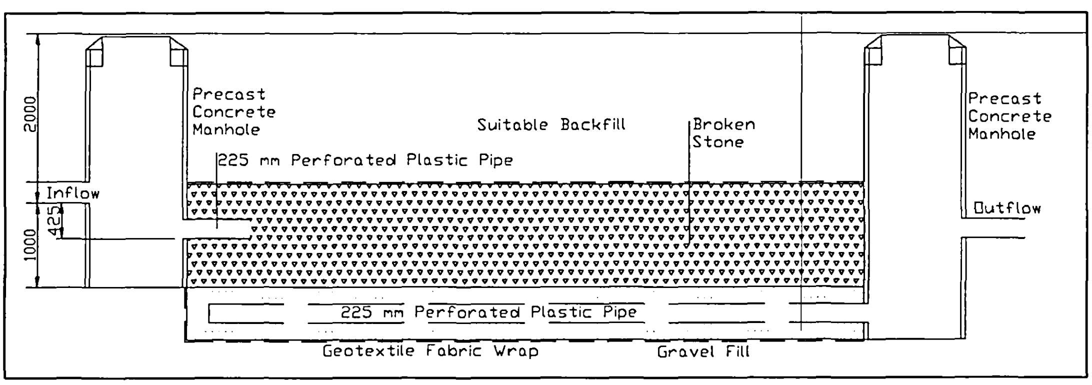



**Figure 6-11: Long section of typical infiltration trench, e.g. Walker Dam**

The length of the system was increased from 8.0 to 9.9 metres to take account of the volume of the downstream monitoring chamber. This was necessary as the downstream chamber uses a high level outlet and therefore provides additional storage volume to the system. The downstream chamber was not explicitly modelled but its volume was incorporated in the trench volume. Table 6-3 shows a list of parameters for the FVD model.

| Parameters [unit] | Example Value | Source of Value |
|---|---|---|
| Hydraulic conductivity of fill material [$ms^{-1}$] | 0.005 | Site Specific. Used measured data |
| Length [m] | 9.9 | Site Specific |
| Width [m] | 2.0 | Site Specific |
| Depth [m] | 1.5 | Site Specific |
| Height of outlet above base [m] | 0.9 | Site Specific |
| Diameter of outlet [mm] | 150 | Site Specific |

**Table 6-3: Site specific data for validation of the FVD model**

More detailed information about the Walker Dam site is provided in Section 3.3.4. and Appendix B. Level recordings in the downstream monitoring chamber were used for model validation and 5 weeks of recorded data is available from February until March 2003 (see Appendix G).

### 6.7.2 Modelling set-up

The system represented by the FVD model has one inlet and one outlet. Figure 6-12 shows a schematic of the flow paths in the FVD model and Figure 6-11 is a long section of the system modelled. The basic code for the FVD model is given in Figure 6-6 and the modified code for this particular system is attached in Appendix E.

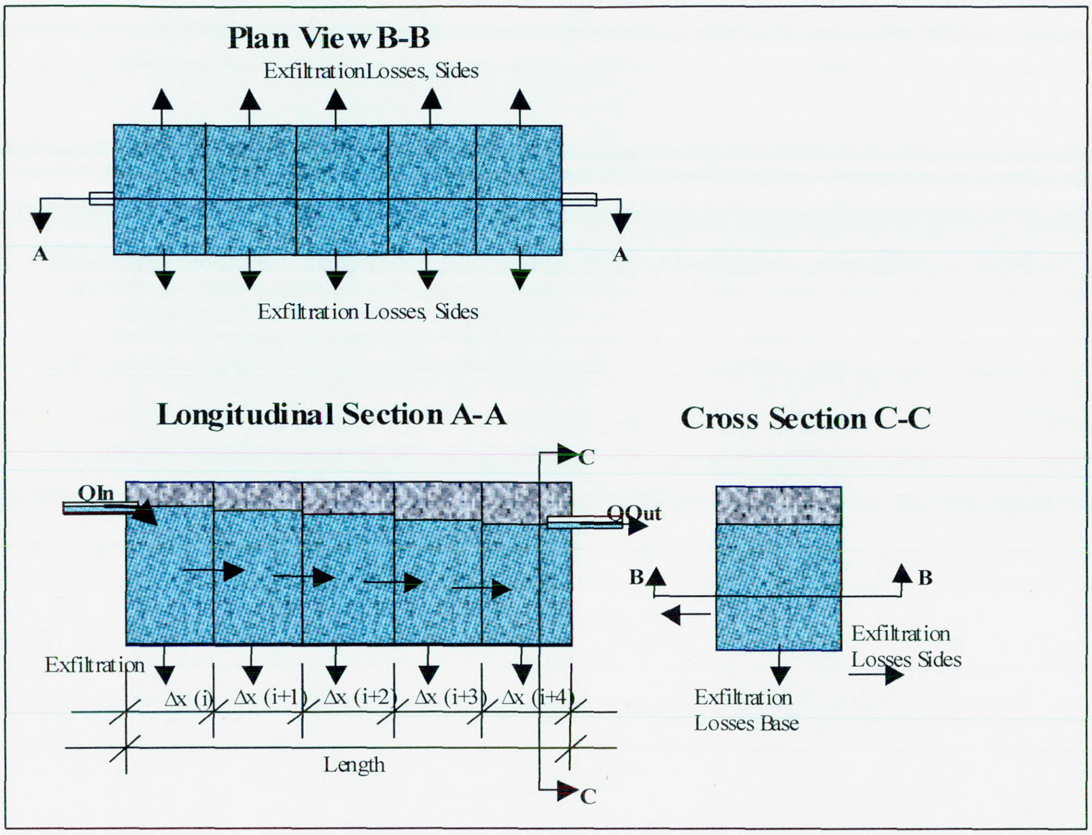



**Figure 6-12: Schematic of flow paths used in the FVD Model**

### 6.7.3 Comparison of recorded and simulated data

A numerical validation, as described in section 5.8.1, was not appropriate as system emptying was slow, resulting in extremely long event durations with only two flow events available for validation. Instead an eye comparison of recorded and computed water level data was undertaken to validate the FVD Model for Walker Dam.

Figure 6-13 to Figure 6-15 show results from this modelling exercise. The great discrepancy during the dry period from 16th February 2003 is due to the monitoring set-up. The water level within the trench is recorded using a Montec Flow Logger, which was secured at the base of the outlet chamber. There was a permanent pool of water in the outlet chamber but the model trench empties during the long dry period.

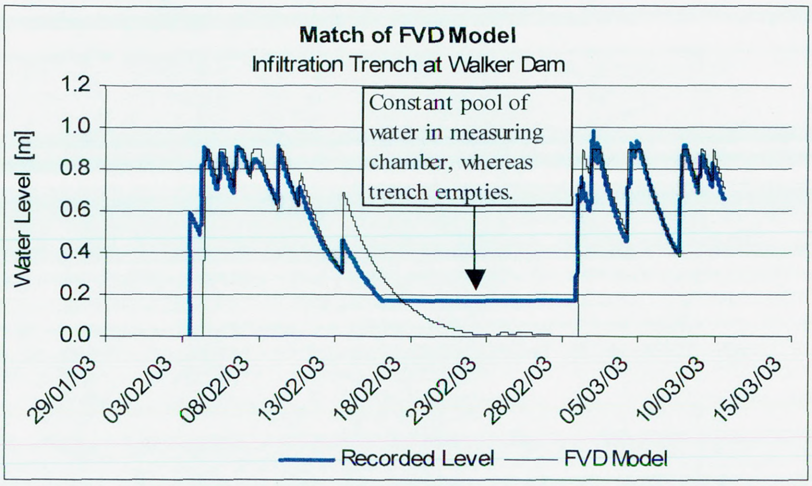



**Figure 6-13: Comparison of recorded and simulated level data at Walker Dam, showing all data**

Figure 6-14 shows level data from the first two weeks. The difference of level at the start of the simulation is due to the difference in saturation of the modelled and monitored system. The model uses a completely dry and empty system, which becomes saturated during the simulation but the monitored system does have some degree of saturation and this causes the late rise of the modelled water level.

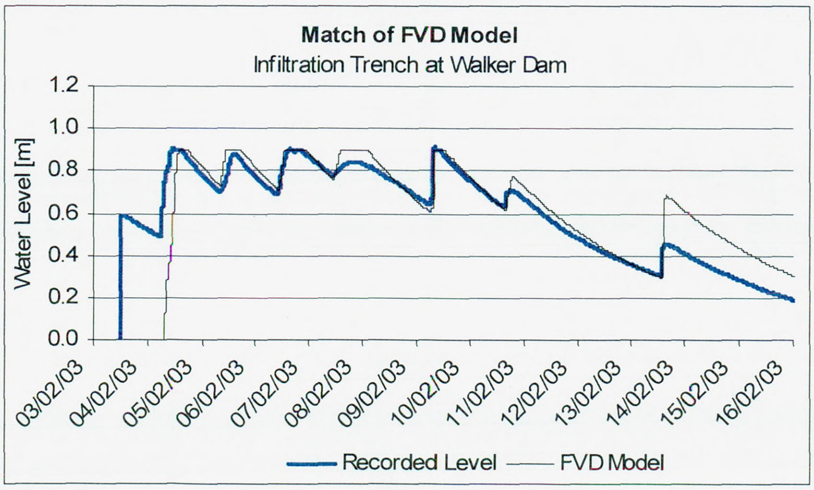



**Figure 6-14: Same as Figure 6-13, showing data from 3rd to 16th Feb 03**

Figure 6-15 shows the last two weeks of the simulation period. The FVD model predicts the water level of the trench well.

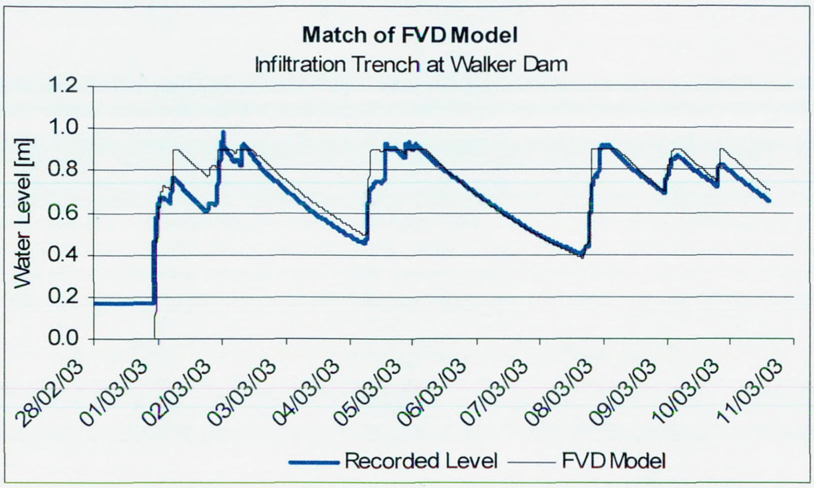



**Figure 6-15: Same as Figure 6-13 showing data from 1st to 11th Mar 03**

The sensitivity analysis showed the influence of the hydraulic conductivity and porosity of fill material (see Figure 6-10). The hydraulic conductivity of the fill material was estimated from the time-lag of the monitored peak flow at the inlet and outlet of the system in relation to the distance of travel. A parallel study, based on an experimental set-up at Riverside Dundee provided measurement data for this parameter, and findings were similar. The only calibration parameters were the porosity of fill material and the exfiltration rates and these were fit to match the monitored outflow. The exfiltration rate of the trench base and sides were set to best represent the declining hydrograph and a good match was achieved. These two parameters were changed with the porosity of the fill material to achieve the best fit of the recorded data using an eyeball comparison of monitored and simulated outflow. Table 6-4 provides a list of these calibration parameters that achieved the best fit.

If no monitored data is available for model calibration, information about exfiltration rates can be obtained from Wallingford Procedure, Soil Maps (The Wallingford Procedure, 1981b) and the porosity of the fill material can be obtained from literature; default values for gravel is 30%.

| Parameters [unit] | Calibrated Value |
|---|---|
| Exfiltration rate of side [$ms^{-1}$] | $2.5*10^{-6}$ |
| Exfiltration rate of base [$ms^{-1}$] | $5*10^{-8}$ |
| Porosity of fill material [%] | 25 |

**Table 6-4: Calibration parameters used for FVD Model**

In addition to the calibration parameters there are additional parameters, which have to be specified and these are listed in Table 6-5

| Parameters [unit] | Example Value | Source of Value |
|---|---|---|
| Time Step [s] | 1 | Check Equation (6-19) |
| Array Elements [-] | 4 | Check Equation (6-19) |

**Table 6-5: Additional parameters used for FVD Model**

## 6.8 Discussion

The newly developed FVD model provides an excellent prediction of flow through gravel filled infiltration trench systems. The FVD model is based on Darcy's law equation and volumetric mass balance. The model was first developed using an Excel spreadsheet and this was translated into a code-based model using the Time Series Manager/ Modelling Package.

A sensitivity analysis was carried out using a fictitious system to identify the general sensitivity of the model towards hydraulic conductivity, porosity and the number of cells of the system being divided. When using as little as two cells the system produces the largest flow volume. The sensitivity reduced with an increase of the number of cells but no significant difference was found between results calculated using more than 10 cells. The optimised number of cells has to be estimated at the beginning of the modelling exercise. Increasing the number of cells improves accuracy but also increases the computational time significantly. Equation (6-19) provides guidance on the highest number of cells appropriate, time step and hydraulic conductivity. The system at Walker Dam was computed using four cells with a length of two metres each. System stability was achieved selecting a time-step of one second. This resulted in a relatively long computational time and data were then computed to two-minute intervals for better data handling using the Hydrol Time Series Manager Database. Depending on the size of the system and the computational power available, larger time steps and longer cell lengths may be appropriate. However, the choice of time step and cell length (or array elements) is always an iterative process which has to satisfy the physical constraints as described in Equation (6-19).

The model showed a linear sensitivity to the change in porosity. With increase in porosity the total flow volume reduces. Although this behaviour appears to be unexpected, it can be explained by having a closer look at the way flow is calculated. An increase in flow is generated from a greater difference in water level from one cell to the next. The reduction of flow due to the increase in porosity results from the reduction in water level (head) in each cell and this is due to the increased storage volume, resulting from the increase in porosity. In reality, material of higher porosity would also be associated with an increase in the hydraulic conductivity, which would result in an expected increase in flow. The sensitivity test also showed an expected increase in total flow with the increase in hydraulic conductivity .

The FVD Model was validated with flow measurements from the infiltration trench at Walker Dam, Aberdeen. The only calibration parameters were the porosity of fill material and the exfiltration rates of base and sides and these were fit to match the monitored outflow. These parameters were changed to achieve the best fit of the recorded data using an eyeball comparison of monitored and simulated outflow., where best findings were produced using a base exfiltration rate of $5.0*10^{-8}ms^{-1}$, a side exfiltration of $2.5*10^{-6}ms^{-1}$ and a porosity of 25%. Depending on the required model accuracy and the site characteristics it may be appropriate to use one value for the base as well as side exfiltration rate. However, it has to be born in mind that the exfiltration rate may reduce over time due to particle accumulation and subsequent clogging. This clogging process is expected to occur at a faster rate at the system base which would result in lower base exfiltration rates.

The hydraulic conductivity was estimated from on-site measurement of inflow and outflow and was set to 0.005 $ms^{-1}$. Measurements from a parallel study of a newly built permeable pavement with gravel fill material were similar and provided confidence in the estimation of the hydraulic conductivity.

The FVD Model is applicable for systems, which can be represented using a one-dimensional approximation. Flow is assumed to be laminar and this restricts its application to systems with a relatively low slope. Further research in form of lab experiments should be undertaken to define the restriction of slope and flow characteristics better.

The FVD model was made available to HRWallingford and will be incorporated in the next release of Infoworks (Version 6.5).

The great advantage of the FVD model is that it computes flow through gravel filled trenches based on their physical characteristics without the need to undertake an extensive calibration process. The main input parameters are the time step and cell length (see Equation (6-19) as well as the physical trench dimensions, porosity of fill material and exfiltration rates. In the absence of field data, exfiltration rates can be estimated from the Wallingford Procedure, Soil Maps (The Wallingford Procedure, 1981b) and the porosity of the fill material can be obtained from literature; default values for gravel is 30%.

# CHAPTER 7 FINDINGS AND CONCLUSIONS

*The following chapter draws together the findings and conclusions from this research programme, which worked towards enhanced detailing and improved operation of in-ground SUDS in Eastern Scotland. Information was combined from a general investigation of 43 different sites and onsite monitoring of 6 typical systems. Computer simulation greatly enhanced the understanding of the hydraulic performance and enabled a performance comparison between the different systems. Furthermore, an improved modelling concept based on the finite-volume-method and Darcy's law (FVD) was developed and validated using on-site-monitored data, and an excellent fit was achieved. Recommended future work is outlined at the end of this chapter.*

## 7.1 Integration of Findings

This research programme worked towards evaluating the effectiveness of in-ground SUDS in the urban environment. This overall aim was achieved by meeting the objectives a to d as outlined in Chapter 1.3. (Results are discussed in more detail in Section 7.2 to 7.7.)

Findings from the visual assessment showed great differences in each system's detailed design, maintenance and operation. These findings were confirmed by on-site monitoring. It appeared that site engineers had little knowledge about in-ground SUDS and planners and designers were unable to perceive fully their performance and operation. The main reason for this shortfall was because of the lack of research, information on the general operation, and the limitations of current models. This study developed a ranking method (see Section 4.4), which provides performance measures from in-situ systems and relates to good and bad examples of detailed design. Verified Erwin models show that individual systems can be simulated successfully provided sufficient monitoring data is available. However, the representation on a catchment scale is currently not available. The new FVD model is introduced to enable the hydraulic representation of in-ground SUDS on a catchment wide scale without the need for extensive monitoring data.

## 7.2 Overall assessment

There is a growing concern in the Scottish Water and SEPA that many systems are permanently blocked and this may never be noticed at locations with overflows. Overflows were found at more than 50% of all systems and more than 30% had signs of temporary blockage. The overall assessment of in-situ systems showed that almost 75% of all systems discharge to natural watercourses, disconnecting a significant amount of impermeable area from combined sewer systems. The drainage areas vary from under 400 $m^2$ up to 200,000 $m^2$ and typically consist of road and roof surfaces from small to medium size housing developments in addition to major roads.

A numerical ranking score was introduced to enable a better identification of good and poor operation. This was undertaken following a flow chart procedure as outlined in Figure 4-3. The scores were given for various criteria relating to the system's water quality, hydraulic performance, detailed design and maintainability and results are outlined in Figure 4-4. Findings showed that almost 50% of all systems were found to be unsatisfactory and more than half of these were rated as failed. 36% provided fair performance and 19% showed good performance. Only one system was considered to be performing excellently.

Several reasons were identified for the poor performance, which are summarised as follows and this meets objective b from Chapter 1.3. Runoff with high sediment loads from unstabilised areas or construction sites was found to be a main problem, impairing the performance and longevity of many in-ground SUDS. Almost 30% of all sites were affected by construction runoff with three systems requiring complete reconstruction, as they had been overwhelmed with high sediment load. At one location construction runoff had blocked the inlet to the trench completely, by-passing all flow (see Plate 4-6). Protecting the drainage inlets until construction and site clean-up were finished and the drained area had stabilised could have prevented these problems. Various techniques are available to stop high sediment load entering storm drains and these are extensively used in the US (USEPA, 2002). Another major issue was the lack of maintenance programmes. This study showed that maintenance was urgently required for most of the systems and that regular maintenance is vital for the longevity of in-ground SUDS. A significant number of systems require major upgrading before they may be considered satisfactory and a maintenance inventory is provided for each system. Poor detailed design was outlined for several systems and this was another main reason for the poor performance overall.

Hydraulic monitoring and modelling of the six typical in-ground SUDS provided detailed information of the performance of each system and this information aided in the assessment of the remaining systems. Extensive monitoring data was required to successfully calibrate each model using the standard modelling package Erwin. The newly developed FVD modelling concept provides modelling capabilities without the need for extensive monitoring data and this will enable a simulation of in-ground SUDS on a catchment wide scale.

## *7.3 Maintenance*

A principal objective of this study was to provide information on the short-as well as long-term maintenance requirements which would enable a satisfactory operation of each systems investigated. During the study, maintenance was carried out at four sites and its effect on each system's performance was assessed. In addition, anecdotal evidence from key personnel in the water authority and highway operators provided valuable information about the maintenance procedures and operation.

The findings from the survey showed that a significant number of in-ground filter drains, infiltration trenches and soakaways require major upgrading before they may be considered satisfactorily. One-off maintenance tasks were identified for 32 sites. The tasks range from a complete reconstruction or substantial replacement to smaller cleaning out tasks (see Table 4-13). In addition to the one-off tasks, ongoing routine maintenance is required for all systems and site specific maintenance intervals are proposed (see Table 4-14). Recommended maintenance intervals vary from annually for major road drainage systems to once in ten years for small systems in housing estates or car parks. Table 4-15 provides an overview of the likely frequency of maintenance activities on in-ground SUDS and Table 4-16 to 4-17 is a detailed maintenance appraisal of all systems studied.

Roadside filter drains may impose a long-term pollution risk to the receiving watercourse and groundwater. Maintenance is undertaken with the primary objective of ensuring hydraulic performance and it is thought that current maintenance techniques allow pollutants to accumulate within the system (see Section 4.8.4). To date, no effective way to extract pollutants from the filter material has been established and once blockage occurs, whole systems have to be replaced. For major trunk roads, replacement of same sections is expected every two years to maintain hydraulic performance. The filter material is replaced only above the drainpipe (see Plate 4-13). This operation allows pollutants to accumulate at the trench bottom, which may impose a long-term risk in the form of ground water contamination. Further research is required to confirm this.

Cleaning techniques were unsuitable for typically trapped gully pots which discharge directly into the filter material of filter trenches (see Section 4.8.4). The most common method is high pressure flushing of the outlet, resulting in mobilising of accumulated particles and extremely high turbidity readings at the system outlet (found at Lang Stracht).

Highway operators and local authorities are required to adopt strict maintenance programs to enhance the performance and increase the longevity of in-ground SUDS. Maintenance programmes were generally not in place but this study provides a maintenance itinerary for each of the inspected sites (see Section 4.8.2 and 4.8.3).

In addition to the lack of maintenance and unsuitable maintenance activities, poor detailing was found to impair the performance of in-ground SUDS and findings are synthesised in section 7.4.

## 7.4 Detailing

The enhancement of knowledge in detailing of in-ground SUDS was another objective of this research and results are summarised here along with a list of recommendations.

The findings showed poor design for some of the more mature sites and this appeared to be due to low confidence in the system's performance. For example, throttles were not installed due to a lack of confidence in the ability of the throttle to operate effectively, or for fear of blockage. The result has been that the storage volume often cannot be utilised effectively and these systems act merely as large storage tanks (Figure 4-10 and Figure 4-13).

High-level by-passes or overflows are used at more than 50% of all system, generally to ensure hydraulic performance in case of extreme rainfall events and to prevent property flooding. However, blockage of in-ground SUDS may never be noticed at locations with overflows as these could be operating constantly. The survey showed that more than 30% had signs of temporary blockage (see Plate 4-4) and one site was found to be completely blocked. The installation of overflows has to be assessed on a site-by-site basis but many installations were found where overflows could have been eliminated as there was no risk of property flooding.

It is thought that disconnecting the inlet from the outlet provides better pollution retention in comparison with systems which discharge directly via perforated drainpipe. Pollutants are filtered out and retained within the filter medium rather then flushed through. However, the reduced hydraulic performance of these systems has to be taken into account. Frequent flooding was discovered at two sites due to the reduced hydraulic capacity (see Plate 4-5 and Plate 4-8(b)).

A few sites were found which did not use any inspection chambers or rodding eyes, which makes these sites impossible to maintain or clean. Other sites could have been improved further by using additional features, such as a dip plate or a rodding eye or the modification of existing details. Often the volume of the sediment sump was not sufficient or the level of the perforated pipe was inappropriate, promoting sediment input into the trench. High level outlet was found to improve the system's performance when situated in good drainage soil by allowing water to be retained within the system for long time periods in poor drainage soil.

Typical trapped gully pots were used at 80% of all sites. Several sites were found where trapped gully pots were filled with debris, causing the inlet to the SUD system to block. It is essential that this risk of blockage is minimised by gully pot emptying. The flow capacity from gully pot outlets, when connected directly into the filter material, was found to be insufficient, resulting in temporary flooding (see Plate 4-5). Offlet kerbs were found to be problematic as inlets to filter drains. This is mainly due to the inability to clean and maintain the inlets. Blocked inlets were found at both sites that used offlet kerbs. Road safety was an issue at one site, where filter material was distributed onto the road (see Plate 4-8).

## 7.5 Hydraulic performance

Monitoring results showed that hydraulic performance of the filter drains were similar to the infiltration trenches (see Section 3.7). Modelling results confirmed these findings and showed that a main reason for the difference in performance was due to the permeability of the soil. However, on-site monitoring also confirmed the variety of findings from the visual inspection and showed that each system had its unique characteristics and limitations. The following provides some of the main findings:

- The infiltration trench at Transy was influenced by groundwater ingress.

- The road side filter drain along Lang Stracht has limited inflow capacity
- The inflow to the trench at Walker Dam also has limited inflow capacity

There are a number of points which can be drawn from these findings. Proper site and ground investigation including measurement of groundwater levels have to be undertaken prior the installation of in-ground SUDS. This will ensure that systems do not intercept groundwater. An aggressive maintenance programme has to be followed to ensure that blockage and reduction in the inflow capacity is minimised.

To provide comparable results, events of more than 12.5mm/h had to be excluded from the analyses for Lang Stracht and events with groundwater ingress were also excluded from the analyses at Transy.

The infiltration trench systems achieved a flow volume reduction of approximately 30 to 70% and peak flow reduction of 47 to 86%. Average lag times were found to be between 27 and 180 minutes. The system at Walker Dam was found to be performing satisfactorily despite long emptying times of up to two weeks and this is in line with findings from Abbott and Comino-Mateos (2001).

The filter drain systems at Spine Road and Glencarse showed a satisfactory flow volume reduction of 37 and 53%, respectively. The system design of the filter drains did not allow monitoring of the inflow and no peak flow reductions could be calculated. The average lag time was between 48 and 82 minutes.

The filter drain system at Lang Stracht performed well for small rainfall events of below 13.2 mm/h with a flow reduction of 71%. However, it is expected that losses due to interception and depression storage have an increased influence for small rainfall events and this is thought to be the main reason for the high flow reduction.

## 7.6 Computer Simulation

Objective c in Chapter 1.3 was met by developing individual hydraulic models for the systems under investigation. Modelling was carried out successfully for all but one of the monitored systems and the findings are outlined here. Transy, could not be simulated successfully in Erwin due to the influence of groundwater ingress. A reasonable to good fit was achieved for all simulated systems with an F-value between 31% and 22% and an $R^2$ between 0.82 and 0.94 (see Table 5-6 and Figure 5-7). Monitored and computed hydrographs for all models are attached in Appendix G.

The treatment volume of both simulated infiltration trench systems was under-designed by 75 to 80% according to standard design (CIRIA, 2000a) (see Table 5-8). Monitoring and modelling results showed that the system at Broxden provided the highest flow volume reduction from all systems. The main reason for the improved performance is due to good drainage-conditions of the soil with infiltration rates of $1.0*10^{-3}ms^{-1}$ (Whitlow, 2001). The increased reduction in flow was supported by the use of small diameter drainpipes which allowed system surcharge, promoting exfiltration (see Appendix G.4). The system emptying time was below one hour using a 10-year design storm and no overflow occurred (see Figure 5-9). Monitoring and modelling of the infiltration trench at Walker Dam gave evidence that the system is located in low permeable soil and system emptying occurred after two weeks for the design storm. Monitoring showed a satisfactory performance despite the long emptying time.

Simulations with recorded rainfall and infiltration rates of less than $1.0*10^{-5}ms^{-1}$ show no significant differences between the systems with blocked base and systems with infiltrating base. Improved performance with up to 12% flow reduction was found for good drainage conditions with infiltration rates between $2.5*10^{-4}ms^{-1}$ and $5.0*10^{-5}ms^{-1}$ for systems with infiltrating base.

Simulation with design rainfall shows that good drainage conditions with infiltration rates of more than $5.0*10^{-4}ms^{-1}$ is needed to significantly reduce the flow.

Permeable soils with high infiltration rates are required to provide significant reduction in percentage outflow when using design rainfall (see Figure 5-10). This is due to the extremely high flow rate produced by the design rainfall and shows that the system at Walker Dam would have little impact during extreme events. A better performance for extreme events may be expected for systems with larger storage volume.

Using recorded rainfall shows that flow volume reduction of more than 50% was achieved in very poor drainage conditions with infiltration rates below $1.0*10^{-7}ms^{-1}$ and this is mainly due to surface runoff losses. Surface runoff losses were found to have a significant influence on small to medium events, but limited influence for extreme events or design rainfall.

## 7.7 FVD model

Chapter 1.3 objective c, required to develop an improved model for a more realistic representation of flow the attenuation through in-ground SUDS. This objective was met by developing the FVD model, which provides an excellent prediction of flow through filter drains and infiltration trenches. The FVD model is based on Darcy's law equation and volumetric mass balance, which incorporates the system's physical characteristics to simulate flow. The FVD Model is applicable to systems, which can be represented using a one-dimensional approximation. Flow is assumed to be laminar and this restricts its application to systems with relatively flat gradients. Further research in the form of laboratory experiments should be undertaken to define better the restriction of slope and flow characteristics (see Section 7.11). The use of different fill material or an alteration of the system dimensions results in a change of flow characteristic. The great advantage in comparison with existing modelling packages is that the physical characteristics of the system are the only input parameters needed to compute the flow. There is no need for large amounts of monitoring data as for the currently used empirical or black box models. Provided dimensions and materials used are known, this model would enable the representation of in-ground SUDS on a catchment scale, enabling planners and design engineers to assess the overall impact of in-situ and future installations of in-ground SUDS.

The FVD Model was validated with flow measurements from the infiltration trench at Walker Dam, Aberdeen, where best findings were produced using a base exfiltration rate of $5.0*10^{-8}ms^{-1}$, a side exfiltration of $2.5*10^{-6}ms^{-1}$ and a porosity of 25%. The hydraulic conductivity was estimated from on-site measurement of inflow and outflow and was set to $0.005\ ms^{-1}$. Measurements from a parallel study of a newly built permeable pavement with gravel fill material were similar and provided confidence in the estimation of the hydraulic conductivity.

The newly developed FVD modelling concept is proposed to be included into Wallingford's popular modelling package, Infoworks. This would enable the assessment of in-ground SUDS on a catchment wide scale.

## *7.8 Recommendations for Design and Detailing*

The philosophy of this research was to synthesise information into improved detailing and operational guidance which could contribute to achieving best long-term performance of in-ground SUDS. Chapter 4 provides results from the visual inspection and section 4.5 and section 4.6 outlines examples of good and poor detailing, respectively. This information has been generalised here to provide a list of recommendations to improve the performance of in-ground SUDS:

a) A sump (wet well) capacity of more than $0.5m^3$ and up to $1.5\ m^3$ to promote sedimentation prior to inflow into the trench system.

b) The chamber inlet and outlet (inlet to trench) should be in level to minimise turbulence and promote sedimentation in the sump prior to inflow into the trench (poor detailing was found at System No 41, showing a 2 metre drop from inlet to outlet).

c) Extensive use should be made of T-pieces and dip plates (skim boards) to hold back any floating particles and chemicals, and reduce the flow velocity, which will improve sedimentation. The T-pieces should be removable to allow access for maintenance and inspection.

d) The inflow and outflow drainage pipes should be disconnected to promote filtration. This will improve the water quality of the outflow as well as promoting flow attenuation (see example of system at Broxden in section 3.3.5).

e) There should be an inspection chamber at either end of the system. This enables system inspection and access for maintenance and cleaning purposes and protection from pollution due to spillage.

f) A monitoring well of 50-150mm diameter perforated pipe should be installed in each infiltration trench, soakaway and for every 100m lengths of filter drain.

g) There should be vehicular access to allow flushing out of debris during maintenance. The installation of rodding eyes may aid access to flush out of debris. Good practice was found at Broxden, where removable plugs were used, which enabled access to the drainage pipes at either end of the system.

h) The perforated inlet pipe should be extended to the downstream manhole to minimise the risk of blockage and increase flow distribution within the system.

i) The elevation of overflows should be maximised to utilised additional storage volume and promote filtration. This will also reduce the number of overflow events and improve the water quality performance.

j) The trench inlet should be near the top of the trench (<0.5m below top) to utilise a maximum of the storage volume.

k) A high-level outlet is recommended for systems with good permeable soils (>1*10-5ms$^{-1}$) to encourage infiltration.

## 7.9 Recommendations for Maintenance and Operation

Section 4.8 provides a maintenance inventory for 32 of the inspected systems and this information is generalised here to provide recommendations for maintenance and operation of in-ground SUDS elsewhere:

a) All drainage inlets that discharge to infiltration systems must be plugged or blocked off during construction to prevent blockage of in-ground SUDS due to high sediment concentration in construction runoff.

b) Pipe systems should be jetted and sedimentation sumps and trapped gully pots emptied, prior to taking in-ground SUDS online. This should be undertaken after construction has ceased and catchment areas stabilised. This may be up to 6 months after construction has ceased. A CCTV survey should confirm cleanliness and proper construction of the drainage system.

c) Level recordings from within the monitoring well should be carried out to confirm that the infiltration system is operating as designed. This shall be undertaken during the first two months from the system being online and also for another month during the wet season (one month from November until April). It is recommended to record levels from within the monitoring well at least twice per week. The installation of a level logger may be advantageous as it provides for continuous level recording and may also reduce the number of site visits.

d) The first inspection should be undertaken within two months after taking the infiltration system online. It should be a visual inspection by opening manholes and inspecting drainage inlets.

e) A maintenance programme should be drawn up for each site during the design stage. One site inspection every six months during the first two years is recommended. This interval may then be reviewed depending on the maintenance requirements.

f) Drainage inlets such as trapped gully pots should be emptied every six months for the first two years. This interval should then be reviewed depending on the amount of accumulated debris. However, it is recommended that trapped gully pots should be emptied at least once per year after leaves have fallen to minimise the risk of blockages.

g) Sedimentation sumps should be emptied prior to taking the system online and again after 1 year of operation. The subsequent maintenance intervals will depend on the amount of accumulated debris during the first year and may vary from 6 months to 5 years.

h) Perforated distribution pipes should be jetted every time the sedimentation sumps are emptied to maintain hydraulic performance and minimise risks of blockage. In addition, jet-water may have to be collected to prevent water pollution of the receiving water.

## 7.10 Recommendations for Modelling In-Ground SUDS

This research produced a specific modelling tool for in-ground SUDS with infiltration. At the time of thesis preparation, it is planned to incorporate this model into the next release of Infoworks software suite. The application of this model is generalised here:

a) Good practise modelling procedure should be following, consisting of calibration and sensitivity analyses and recommendations for the FVD model are provided here.

b) The FVD model was made available to HRWallingford and will be incorporated in the next release of Infoworks (Version 6.5).

c) My research has shown that Equation (6-19) is applicable to determine the highest number of cells appropriate, time step and hydraulic conductivity and this is applicable to the FVD model. The physical interpretation of Equation (6-19) is that when the inflow is larger than outflow the net water increase in the cell cannot exceed the cell capacity per time step; when the cell is losing water because outflow is larger than inflow, the net water decrease is not allowed to over-dry the cell per time step. It is proposed that the computation of time-step and number of cells will be automated within Infoworks.

d) It is recommended to set the default time step to 30 seconds and $\Delta x$ to 2 metres and that initial conditions are calculated to check that Equation (6-19) is satisfied. If Equation (6-19) is not satisfied it is recommend to automatically decrease the time step by an increment of 5 seconds until the model stability is achieved.

e) $\Delta x$ can also be adjusted, depending on the accuracy required. It is recommended that $\Delta x$ should not be set below 0.1m due to the increase in computational time which would result.

f) Tests should preferably be carried out to determine the porosity and conductivity of fill material in order to provide specific values for different fill materials, which will be applicable from the Infoworks help wizard.

g) If no other information is available it is recommended to use a porosity of fill material of 30% and a hydraulic conductivity of fill material 0.005 $ms^{-1}$.

h) This guidance will be available in the Infoworks help wizard.

## 7.11 Future Work

The following areas of study are recommended for future work:

1. This study showed many systems with poor detailed design and it is suggested that findings will be used as basis for developing a guidance document on enhanced detailed design, maintenance procedures and operation of in-ground SUDS. Findings from this study are proposed to be included in further research funded by the Scottish Executive and outcomes of this project will be synthesised into a guidance document.

2. This study focused on the effectiveness of in-ground SUDS and outcomes and conclusions were based mainly on the system's hydraulic performance and visual observations. A detailed water-quality monitoring programme would be required to identify the optimal hydraulic performance whilst providing sufficient pollutant retention, and this is important for locations with stringent water quality consent.

3. Infiltration of runoff from major roads into the subsoil is prohibited in some countries due to the risk of ground water pollution (e.g. Germany). Further work should be undertaken to identify the risk of groundwater pollution from road runoff and better understand its long-term environmental impact.

4. Innovative techniques and maintenance procedure are required to improve the operation and longevity of in-ground SUDS and to enable the pollution extraction from in-situ systems. Future work should be carried out starting on pilot or laboratory studies and then continue onto field experiments with on-site monitoring of flow and water quality. Improved maintenance procedures are urgently needed to enable satisfactory long-term performance, in particular with regards to water quality. This research showed that maintenance was primarily undertaken to improve the hydraulic performance. Water quality pollution was discovered due to inappropriate maintenance techniques.

5. Additional research should be undertaken on monitoring the long-term accumulation of sediments and associated pollutants. In association with this monitoring programme,, a comparative study of sites with and without maintenance programmes would provide valuable information on the effectiveness of maintenance activities and this could lead to improved maintenance operations.

6. Darcy's law was derived from steady state flow, but was proven to be valid to model unsteady flow as long as the flow is in laminar condition. If the flow condition becomes turbulent the modified Darcy's law method has to be used. Further research in form of laboratory experiments is proposed to enable the calibration of the FVD model for turbulent flow conditions. This laboratory experiment could be extended to identify the influence of the base gradient and the type of fill material.

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# APPENDICES

# Appendix - A Publications

*Appendix A provides a selection of papers that were published during the course of this research that substantially contributed to the knowledge enhancements and abstracts are included for easy reference.*

***Appendix - A.1 Schlüter, W. Jefferies, C. (2005). The real issues with in-ground Sustainable Urban Drainage Systems in Scotland. Accepted for publication at 10th International Conference on Urban Drainage, Copenhagen, Denmark,***

***Appendix - A.2 Schlüter, W. and Jefferies, C. (2004). Monitoring and modelling of three different in-ground SUDS in the East of Scotland. In Proc: NOVATEC 5th International Conference on Sustainable Techniques and Strategies in Urban Water Management, Lyon – France.***

***Appendix - A.3 Schlüter, W. Spitzer A, and Jefferies C. (2002). Performance of 3 Sustainable Urban Drainage Systems in East Scotland. In Proc: 9th International Conference on Urban Drainage, Portland Oregon USA.***

***Appendix - A.4 Schlüter, W. Jefferies, C. (2002). Modelling the Outflow from a Porous Car Pavement. Urban Water Journal, Vol 4/3, pp 245-253, Elsevier Science Ltd.***

***Appendix - A.5 Schlüter, W. Jefferies, C. (2001). Monitoring the Outflow from a Porous Car Park. In Proc: First National Conference on Sustainable Urban Drainage, Coventry.***

# THE REAL ISSUES WITH IN-GROUND SUSTAINABLE URBAN DRAINAGE SYSTEM IN SCOTLAND.

Wolfram Schlüter[1] and Chris Jefferies[2]

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[2] *Urban Water Technology Centre, University of Abertay Dundee, Bell Street, Dundee, DD1 1HG, Scotland*

**EXTENED ABSTRACT**

Infiltration trenches and filter drains are the most common types of sustainable urban drainage systems (SUDS) in Scotland. Despite their extensive use there has been only limited examination of their performance with general expectation that failure through lack of maintenance and poor detailing design would necessitate reconstruction within a limited time period (Warnaars, 1999, Abbott, 2001 and Pratt, 2001).

The research worked towards enhanced detailing and improved operation of in-ground SUDS. It focused on information gained from on-site monitoring of three filter drain and three infiltration trench systems and combined the outcomes with information gathered from some 40 assessments of in-situ systems in Eastern Scotland. This survey will be extended to 65 site assessments.

The study has covered on systems which jointly convey runoff from roads and domestic properties. Consequently the work is of interest, both to water companies and highway authorities. The paper outlines the following points:

- Performance overview of infiltration trench and filter drain systems in Eastern Scotland
- Assessment of maintenance activities undertaken at selected sites
- Appraisal of activities to restore the system's condition and to maintain it in a satisfactory state
- List of recommendations to enhance the detailing and operation of in-ground SUDS

Current findings showed that almost 50% of all systems were found to be unsatisfactory and more than half of these were rate as failed. 36% provided fair performance and 16% showed good performance. Only one system was considered to be performing excellently. Several reasons were identified for the poor performance. The main problem was found to be due to runoff with high sediment loads. Runoff from unstabilised areas or construction runoff was found to be affecting the longevity of in-ground SUDS. Almost 30% of all sites were affected by construction runoff. Construction runoff at one location had blocked the inlet to the trench completely and all flow bypasses the filter as shown in Plate 1.

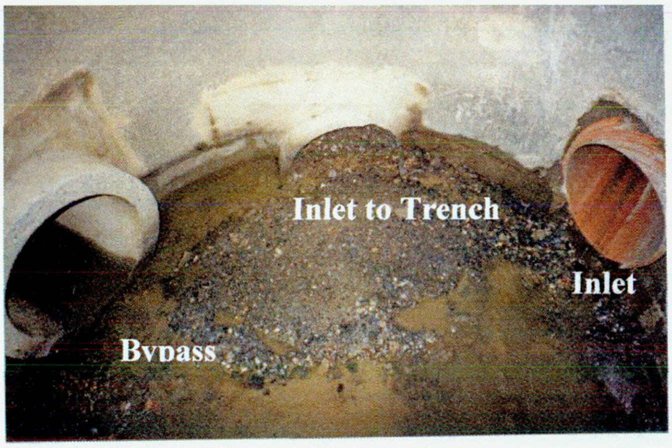


**Plate 1: Completely blocked inlet to trench from construction runoff**

This survey showed that almost 75% of all systems discharge to natural watercourses. High-level bypasses or overflows are used to ensure hydraulic performance in case of extreme rainfall events. There is a growing concern in the water authority and SEPA that many systems are permanently blocked, which may never be noticed at locations with overflows. Overflows were found at more than 50% of all systems and more than 30% had signs of temporary blockage.

Maintenance programs were generally not in place but this study showed that regular maintenance is vital for the longevity of in-ground SUDS. A significant number of systems require major upgrading before they may be considered satisfactory and a maintenance appraisal is provided for each system.

# Monitoring and modelling of three different in-ground SUDS in the East of Scotland

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**ABSTRACT**

This paper outlines results from onsite monitoring of three different in-ground SUDS, and modelling results from rainfall runoff simulations using the stormwater modelling program Erwin. Two systems are infiltration trenches serving relatively small housing estates, one at Broxden, Perth and the other at Walker Dam, Aberdeen. The third system is a typical roadside filter drain, located along the A944 in Aberdeen. Excellent flow attenuation was monitored at Broxden with good flow prediction from hydraulic modeling. Flow monitoring at Walker Dam showed reasonable attenuation, and flow prediction improves after the system was cleaned out. Monitoring of the roadside filter drain showed hydraulic failure due to frequent overspill of the gully pots. Hydraulic simulation of a limited period of six months provides evidence of these findings.

**Keywords:** Hydraulic, filter drain, infiltration trench, simulation, SUDS

9th International Conference on Urban Drainage, Portland - USA
8th -13th September 2002

# Performance of Three Sustainable Urban Drainage Systems in East Scotland

Wolfram Schlüter, Adolf Spitzer and. Chris Jefferies

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**ABSTRACT**

The performance of three different Sustainable Urban Drainage Systems (SUDS) in East Scotland is outlined in this paper. These systems are a porous pavement at the Royal Bank of Scotland (RBS) in Edinburgh, a roadside filter drain along Lang Stracht Aberdeen (LSA) and a regional SUDS at Dunfermline Eastern Expansion (DEX). The systems' performance in attenuating flows proved to be satisfactory. This is shown by consideration of percentage runoff, initial runoff loss, monthly outflow reduction of rainfall and lag time. The systems were found to perform well in attenuating pollutant peaks. This is investigated through comparison of inflow and outflow concentrations as well as comparison with Environmental Quality Standards (EQS). Samples were analysed for standard sanitary parameters at all sites in addition to heavy metals and hydrocarbons at RBS.

A computer model was developed to simulate outflow from the porous pavement at RBS and the filter drain at LSA. The modelling results show an excellent prediction of the outflow behaviour from the porous pavement and preliminary simulation show reasonable agreement with the outflow from the filter drain. Research at LSA and DEX is ongoing.

**Keywords:** Filter drain, hydraulic performance, porous pavement, regional SUDS, sustainable urban drainage, water quality

Urban Water Journal, Vol 4/3, pp 245-253, Elsevier Science Ltd, 2002

# Modelling the Outflow from a Porous Pavement

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**ABSTRACT**

The pervious pavement from industrial and commercial areas is a form of SUD system which has been developed to reduce runoff flow rates and volumes. This study reports on the modelling of the 20-month-old porous pavement at the Royal Bank of Scotland Headquarters in Edinburgh. A hydraulic model was developed using the Stormwater Modelling Program **R▪*Win***. Fifteen events were available for use in the calibration and verification procedure. The modelling results has shown an excellent prediction of the outflow behaviour of the site investigated.

**Keywords:** Porous pavement, modelling, calibration, verification, hydraulic, infiltration, flow attenuation.

First National Conference on Sustainable Urban Drainage, Coventry
June 2001

# Monitoring the outflow from a Porous Car Park

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**ABSTRACT**

Pervious pavements have been developed as source control systems to attenuate flows and provide a measure of pollution control in urban areas while at the same time preserving a relatively intensive land use. However, the attenuation and pollution reduction potential of the systems is relatively unknown, and there is concern about soil and groundwater contamination. This paper reports on two field studies of pervious pavements used as car parks in Edinburgh which are four and two years old respectively. Both car parks were constructed using Formpave blocks and are constructed in locations were exfiltration is either insignificant or not possible. The monitoring programme measured hydrological, hydraulic and water quality parameters. The hydrological monitoring produced consistent data on flow attenuation, water retention/ disposal and a comparison with runoff from an impermeable car park. Analysis of both spot and event based samples shows good outflow quality, and results from heavy metals analyses shows very favourable comparison with drinking water standards. Hydrocarbons data confirmed that some residue from oils and fuels may not be retained by the pavement structure.

**Keywords:** Pervious pavement, sustainable, hydrology, water quality.

# Appendix - B Site Information of Systems Monitored

## B.1 Filter drain along Lang Stracht, Aberdeen
*B.1.1 Site Description*
*B.1.2 Monitoring Information*

## B.2 Filter drain along Spine Road, Dunfermline
*B.2.1 Site Description*
*B.2.2 Monitoring Information*

## B.3 Filter drain along A90 near Glencarse, Perth
*B.3.1 Site Description*
*B.3.2 Monitoring Information*

## B.4 Infiltration Trench at housing estate Walker Dam, Aberdeen
*B.4.1 Site Description*
*B.4.2 Monitoring Information*

## B.5 Infiltration Trench at housing estate Broxden, Perth
*B.5.1 Site Description*
*B.5.2 Monitoring Information*

## B.6 Infiltration Trench at housing estate Transy, Dunfermline
*B.6.1 Site Description*
*B.6.2 Monitoring Information*

## *Appendix - B.1 Filter drain along Lang Stracht, Aberdeen*

### B.1.1 Site description

The filter drain system is located alongside Lang Stracht, a section of the A944 in Aberdeen where widening and resurfacing was carried out during spring of 1999. The catchment area consists of a tarmac road and footpath surface, and was measured at 9,502 $m^2$. The Annual Average Daily Total (AADT) Vehicles on Lang Stracht was 7900 Eastbound and 7250 Westbound of which 3% were Heavy Goods Vehicles (HGV's). Plate B-1 shows a site view of Lang Stracht.

**Plate B-1: Site view of filter drain along Lang Stracht**

### B.1.2 Monitoring information

Monitoring at Lang Stracht was undertaken for 34 months from January 2000 until October 2002. Flow and quality data were obtained from a gully inflow at Catch Pit 4, as well as from the filter drain outflow at Catch Pit 10. Figure B-1 shows the site layout and monitoring locations. Water quality data was monitored during targeted events using quality sondes, which provide continuous data in addition to event based water samples. Outflow data was monitored using a Sigma 950, which was later replaced with a Sigma 911. Inflow was recorded with a tipping bucket gully monitor. In addition, both locations were fitted with a Vegason level monitor.

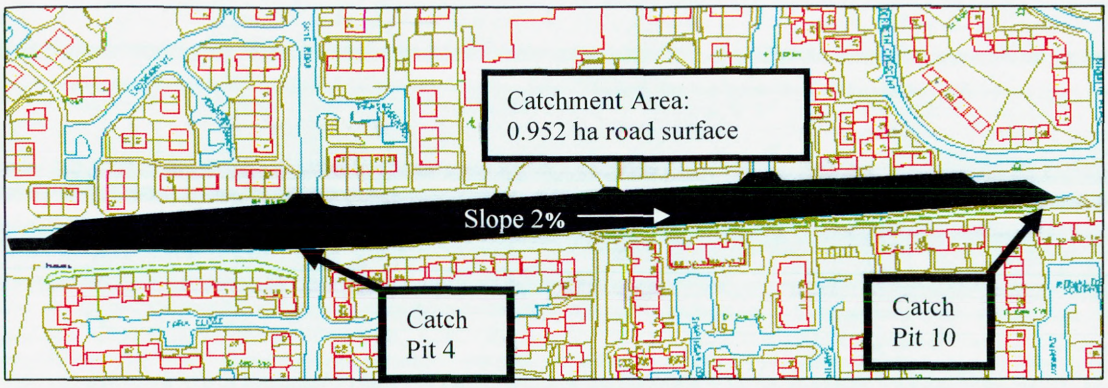



**Figure B-1: Lang Stracht site layout and monitoring location**

Figure B-2 is a bar chart of the monitoring period and available data obtained from each of the instruments.

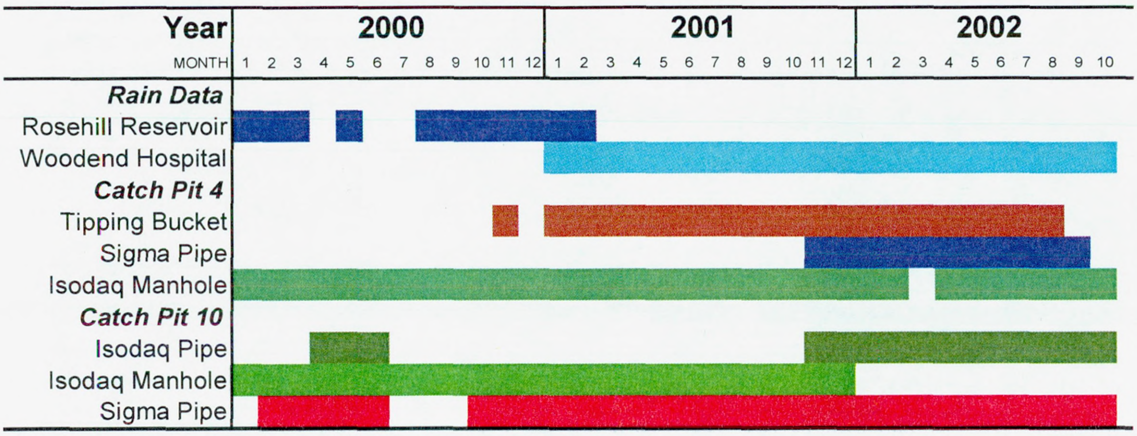



**Figure B-2: Bar chart of monitoring period**

More than one hundred flow events were recorded at Lang Stracht. Hydraulic failure was discovered at the end of the monitoring period due to overspilling of gully pots during rainfall events of 13.2mm/h. No reliable information can be given for events of more than 12 mm/h and flow analyses were carried out for the remaining 87 events. Peak outflow varied from 11.1 l/s to 0.1 l/s producing a total volume of 25.3 and 0.1 mm, respectively. The filter drain system reduces the flow volume by over 70%. Lag time varies from just over one hour to 17.7 h. Table B-1 provides a summary of recorded events at Lang Stracht.

| | Rain data | | | Outflow Flow | | | |
|---|---|---|---|---|---|---|---|
| | Event Duration [d:hh:mm] | Total Rain [mm] | Peak Rain [mm/h] | Total Flow [mm] | Peak Flow [l/s] | Flow Vol [%] | Lag Time [hh:mm] |
| Min | 0:01:08 | 2.0 | 1.2 | 0.1 | 0.1 | 4% | 01:10 |
| Max | 16:12:56 | 61.8 | 12.0 | 25.3 | 11.1 | 81% | 17:40 |
| Mean | 1:22:23 | 10.2 | 6.6 | 3.6 | 1.6 | 29% | 07:20 |

**Table B-1: Summary of events recorded at Lang Stracht**

Flow monitoring at Lang Stracht was carried out at two monitoring locations. The upstream monitoring location was used to record a change in water level, but no flow monitoring was undertaken. Figure B-3 shows the head discharge relationship from the system's outlet and Appendix C.1.2 shows typical hydrographs from Lang Stracht.

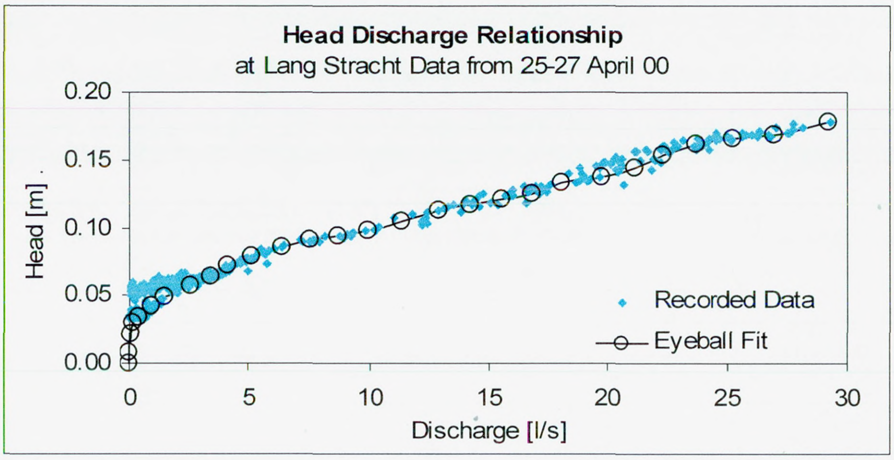



**Figure B-3: Head discharge relationship, Lang Stracht**

## Appendix - B.2 Filter drain along Spine Road, Dunfermline

### B.2.1 Site description

The filter-drain system is located alongside Spine Road South/ Central, within the Dunfermline East Expansion Area (DEX). The system's catchment area consists of road and footpath surfaces in addition to the verge and the filter drain itself and was measured at 3,058 $m^2$. Plate B-2 shows a view of the filter drain along Spine Road.


**Plate B-2: Site view of filter drain along Spine Road**

Details of the filter drain are included in Appendix D.2. Perforated pipe sizes which vary from 150 mm to 225 mm and are located approximately 200 mm above the base of the trench. The filter material consists of 20 mm single sized washed gravel, which is enclosed in geo-textile for separation from the surrounding soil. The gully pots are located along side the road in intervals from 8 to 30 metres.

### B.2.2 Monitoring information

Monitoring at Spine Road was undertaken for 4 months from March 2003 until June 2003. Flow and quality data were obtained from two inspection chambers. The whole catchment area totals to 3058 $m^2$ and this contributes to the outlet, the downstream monitoring location. The second monitoring location is located some 90 m upstream, which receives runoff from an area of 2011 $m^2$. Water quality data was monitored continuously using quality sondes set at 15-minute interval. Flow data was monitored using two Sigma 950 in 5-minute interval. Rain data was monitored continuously in two-minute intervals using a

Casella raingauge. The raingauge was located approximately 1km Northwest of the monitoring location at Spine Road.

Figure B-4 is a bar chart of the monitoring period and available data obtained from each of the instruments.

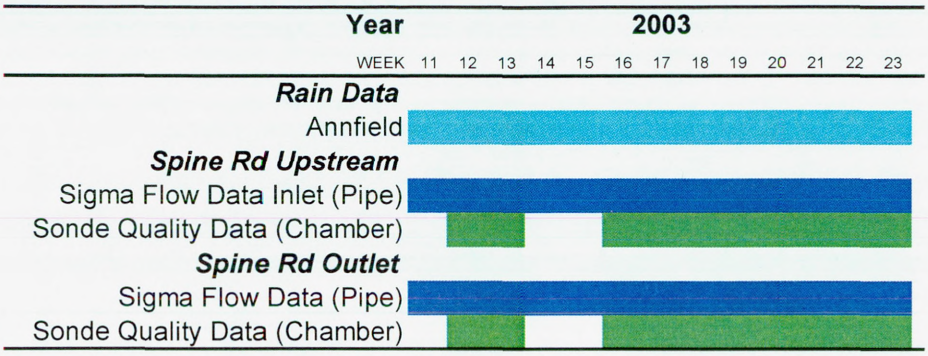



**Figure B-4: Bar chart of monitoring period at Spine Road**

Ten flow events were recorded at Spine Rd, and peak flow varies from 5 l/s to 0.5 l/s at the upstream monitoring location and 18 to 1 l/s downstream. No percentage runoff could be calculated at this site as both monitoring locations receive flow that had passed through the filter material and flow reduction had already occurred (see Section 3.6.5).

The mean reduction of flow volume between the upstream and downstream monitoring location was 73% and 37%, respectively. This corresponds to a peak flow increase of 121% on average. The time lag between the two monitoring locations varied from 27 to 211 min and averaged at 82 min. A summary of recorded events is given in Table B-2. One event was recorded with a flow volume of greater than 100% and the main reason is thought to be spatial variation of rainfall.

**Table B-2: Summary of events recorded at Spine Road**

| | Rain data | | | Upstream Flow | | | Downstream Flow | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Duration [d:hh:mm] | Total [mm] | Peak [mm/h] | Total Flow [mm] | Peak Flow [l/s] | Flow Vol [%] | Total Flow [mm] | Peak Flow [l/s] | Flow Vol [%] | Lag Time [hh:mm] | Peak flow increase [%] |
| Min | 0:03:18 | 3.0 | 3.0 | 0.46 | 0.49 | 14% | 1.59 | 1.24 | 31 | 00:27 | 26% |
| Max | 2:23:44 | 37.6 | 30.0 | 15.21 | 5.13 | 64% | 31.62 | 18.20 | 110 | 03:31 | 255% |
| Mean | 0:23:22 | 9.7 | 10.5 | 2.87 | 1.74 | 27% | 6.55 | 3.96 | 63 | 01:22 | 121% |

The main difference between the system discharging to the up- and downstream monitoring location was the longitudinal slope of the drained area and the filter system. This is thought to be the main reason for the difference in performance. The slope of the upstream section is 1.3% in comparison to 3.5% for the downstream section. The lower slope allows for a more attenuated discharge, which utilises more storage volume and allows for a greater exfiltration. The higher slope promotes an increase in flow velocity with less attenuated flow and greater discharge.

Flow monitoring at Spine Road was carried out at two monitoring locations and both head discharge relationships were similar. Figure B-5 shows the head-discharge relationships and Appendix C.2.2 shows typical hydrographs monitored at Spine Road.

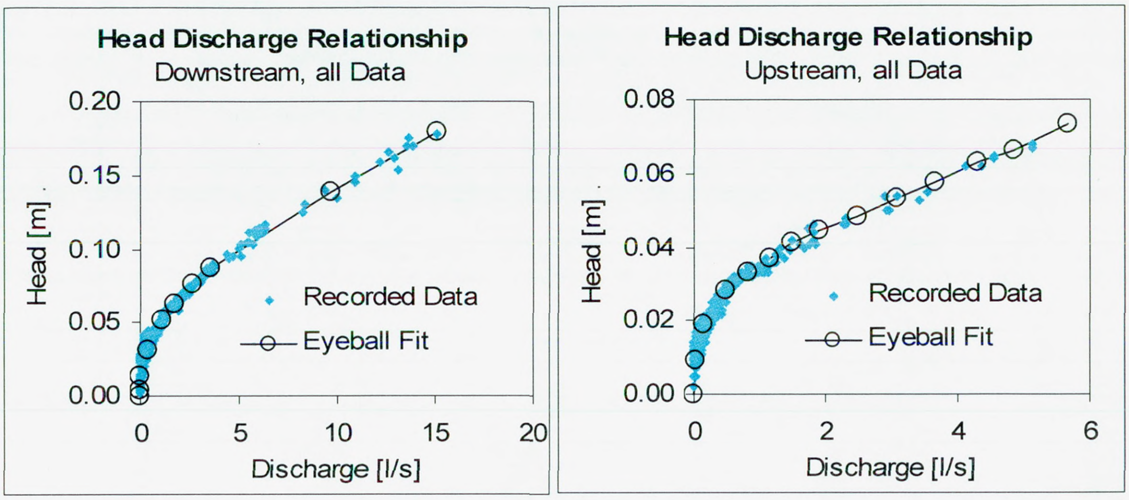



**Figure B-5: Head-discharge relationship, Spine Road, upstream & downstream**

## Appendix - B.3 Filter drain along the A90, Glencarse

### B.3.1 Site description

This filter-drain system was located alongside the A90 dual carriageway near to Glencarse between Dundee and Perth. It is a typical filter system receiving lateral sheet flow from the dual carriageway. The system's catchment area consists of road surfaces from the dual carriageway in addition to a small lay-by section located upstream of the filter drain and was calculated at 7100 $m^2$. A perforated drainpipe was located near the base of the filter drain. Plate B-2 is a view of the filter drain.

**Plate B-3. Site view of filter drain along A90, Glencarse**

Details of the filter drain are included in the Appendix D.3. Perforated pipe sizes vary from 225 mm to 300 mm and are located approximately 200 mm above the base of the trench. The filter material consists of crushed rock, which is enclosed in geo-textile for separation from the surrounding soil. The trench receives surface water as lateral sheet runoff along the system's length. Inspection chamber are located in 100 metre intervals.

### B.3.2 Monitoring information

The filter drain was monitored for six months from June 2003 until December 2004. Figure B-6 is a bar chart of the monitoring period. Flow data was monitored at two locations, the inflow and outflow of the infiltration trench (see Appendix C.3 and D.3). Flow data was measured using Sigma 950 flow loggers and rain data was recorded continuously in two-minute intervals using a Casella raingauge. The raingauge was located adjacent to the system.

Flow monitored at the upstream location showed a good response to the rainfall but the downstream location monitored flow during only one event. A dye test at the end of the monitoring period indicated that flow was bypassing the downstream monitoring location. Flow enters the receiving water course via a local ditch which receives filtered runoff from the SUD system. Therefore performance analyses can only be carried out for upstream part of the system.

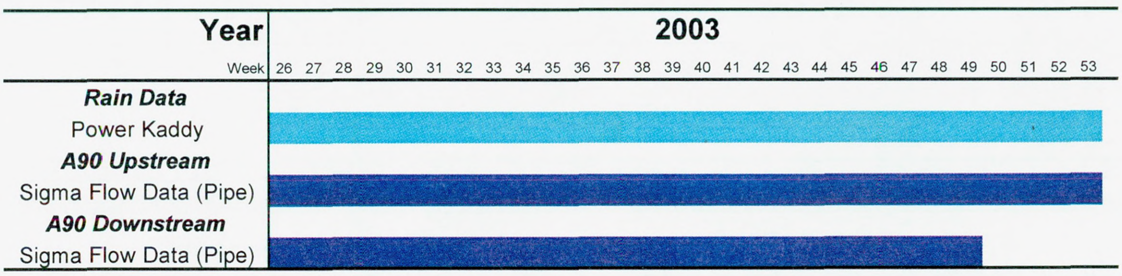



**Figure B-6: Bar chart of monitoring period at A90, Glencarse**

Nineteen flow events were recorded at the upstream monitoring location and peak flow varied from 21 l/s to 0.5 l/s. No percentage runoff could be calculated at this site as the monitoring location received flow that had passed through the filter material and flow reduction had already occurred. The max, min and mean reduction of flow volume were 93%, 5% and 53% respectively. The mean time lag at the upstream monitoring location was 48 min ranging from 9 to 103 min. Table B-3 gives a summary of recorded events at Glencarse. A complete list of events is given in Appendix C.3.

| | Rain data | | | Upstream Flow | | | |
|---|---|---|---|---|---|---|---|
| | Duration [d:hh:mm] | Total [mm] | Peak [mm/h] | Total Flow [mm] | Peak Flow [l/s] | Flow Vol [%] | Lag Time [hh:mm] |
| Min | 0:00:14 | 2.00 | 2.00 | 0.20 | 0.46 | 7 | 00:09 |
| Max | 0:16:34 | 13.00 | 72.00 | 11.78 | 20.94 | 95 | 01:43 |
| Mean | 0:04:18 | 5.53 | 11.05 | 3.18 | 3.22 | 47 | 00:48 |

**Table B-3: Summary of events recorded at A90 Glencarse**

Only the upstream section received flow and could be used to develop a head-discharge relationship. One event was monitored where flow exceeded 5 l/s and most data points were below 2 l/s. There was also a permanent water level of 0.04 m, which was due to the slope and level of the drainpipe. Figure B-7 shows the head-discharge relationship at Glencarse and Appendix C.3.2 shows typical hydrograph.

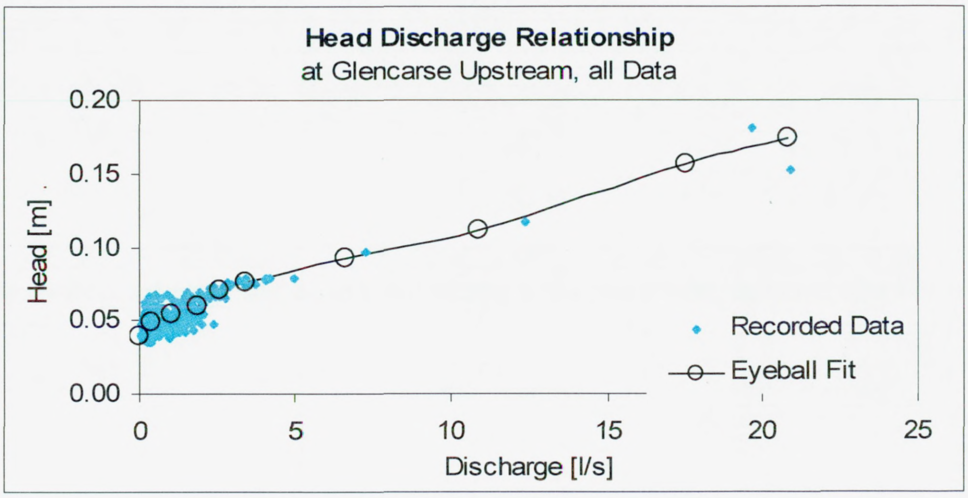



**Figure B-7: Head discharge relationship, Glencarse upstream**

## *Appendix - B.4 Infiltration trench at housing estate Walker Dam, Aberdeen*

### B.4.1 Site description

The surface water drainage at Walker Dam serves roof and road surfaces of 15 houses on an area of $7000m^2$. Road runoff is intercepted via typically trapped gully pots and then conveyed to the infiltration trench. Purified storm water is discharged into the Walker Dam Lake. No design calculation is available for this site. Plate B-4 shows a site view of Walker Dam, which was constructed in spring 2000.

**Plate B-4: Site view of infiltration trench at Walker Dam**

Appendix D. 4 shows a detailed construction drawing of the infiltration trench at Walker Dam. The system is 8 m long, 2 m wide and 1.9 m deep. It is designed as infiltration system using a total storage volume of approximately 9.1 $m^3$. The gravel fill material is separated from the surrounding soil using a geo-textile. There are two inspection chambers, one upstream and one downstream of the trench and these are built as sediment traps. The inflow into the trench is distributed via a 1.0 m long 225 mm diameter perforated pipe. Flow is then attenuated within the stone-fill structure. There are two layers of stone fill material. The upper layer consists of broken stone and the lower layer of gravel fill material. The gravel fill incorporates a 225 mm diameter perforated pipe, which is located just above the base of the system. This pipe discharges purified flow to the second sediment chamber. The system's outflow pipe is elevated by 1.2 m utilising additional storage-volume within the sediment chambers.

### B.4.2 Monitoring information

The infiltration trench at Walker Dam was monitored for 3½ months from November 2002 until February 2003. Figure B-8 displays a bar chart of the monitoring period. Flow data and water quality data was obtained at two monitoring locations, the inflow and outflow of the infiltration trench (see Appendix D.4). Water quality data was monitored using quality sondes providing continuous data and flow data was measured using Sigma 911 flow loggers. Rain data was monitored continuously in two-minute intervals using a Casella raingauge. The raingauge was located approximately two km North of the Walker Dam housing estate, on premises of Woodend Hospital.

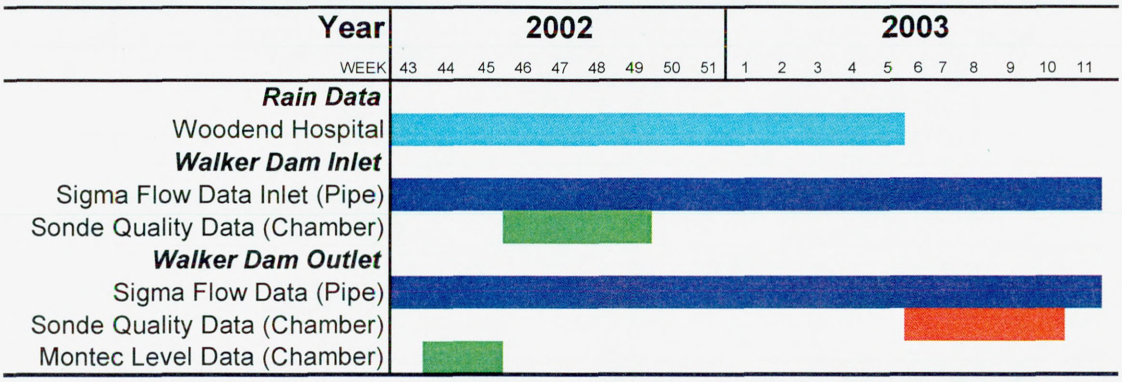



**Figure B-8: Bar chart of monitoring period at Walker Dam**

38 flow events were recorded at Walker Dam and peak inflow varies from 34 l/s to 2 l/s producing a total volume of 1.3 and 20.6 mm, respectively. The percentage runoff at this site is 81% on average. The infiltration trench reduces the inflow volume by 34% with a mean peak flow reduction of 47%. Lag time varies from one minute to 90 minutes with a mean rainfall duration of almost 9 hours. Eight events were recorded where inflow exceeded 100% and it is thought that this is mainly due to the spatial variation of rainfall. Five events were recorded with more than 100% outflow and this is thought to be due to measurement errors, which increased due to the steep slope of the system outlet. Table B-4 provides a summary of recorded events at Walker Dam.

| | Rainfall | | | Trench Inflow | | | Trench Outflow | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Duration [d:hh:mm] | Total [mm] | Peak [mm/h] | Total Flow [mm] | Peak Flow [l/s] | Flow Vol* [%] | Total Flow [mm] | Peak Flow [l/s] | Flow Vol [%] | Lag Time [hh:mm] | Peak flow reduction [%] |
| Min | 0:01:22 | 2.0 | 1.2 | 1.27 | 1.95 | 27% | 0.70 | 0.81 | 30 | 00:01 | -16% |
| Max | 1:13:04 | 18.8 | 42.0 | 20.58 | 34.04 | 124% | 15.84 | 9.12 | 123 | 01:23 | 81% |
| Mean | 0:08:43 | 7.7 | 10.0 | 6.61 | 7.62 | 81% | 4.27 | 3.67 | 66 | 00:27 | 47% |

**Table B-4: Summary of events recorded at Walker Dam**

Outflow monitoring at this location was particularly difficult, as the system's outflow is via a high level outlet, which has a steep slope. Velocity readings at this location were very high and this resulted in a relatively wide scatter of data points. Figure B-9 shows the head-discharge relationship at Walker and Appendix C.4.2 shows typical hydrographs. System emptying time was up to two weeks.

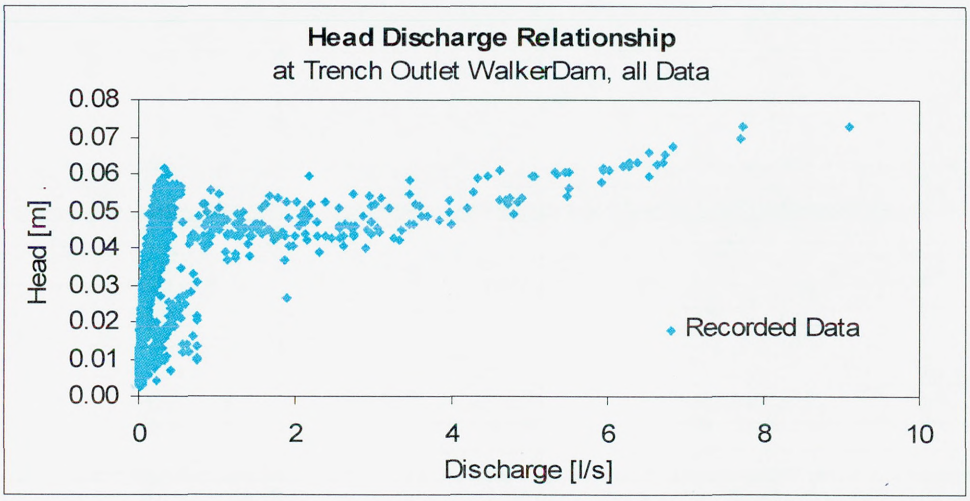



**Figure B-9: Head-discharge relationship, Walker Dam**

## Appendix - B.5 Infiltration trench at housing estate Broxden, Perth

### B.5.1 Site description

The surface water drainage serves 14 houses on an area of 7500 $m^2$ including road and roof surfaces. Road runoff is intercepted via typically trapped gully pots and then conveyed to the infiltration trench. Purified stormwater is discharged into the Craigie Burn. Plate B-5 shows a site view of the infiltration trench at Broxden, Perth, which was constructed in summer 1999. The system is designed for a 1-year storm event.

**Plate B-5: Site view of infiltration trench at Broxden**

Appendix D.5 shows a detailed construction drawing. The upper end of the infiltration trench incorporates a 150 mm diameter overflow. The infiltration trench is 45 m long and 1.2 m deep and 0.75 m wide. The system is designed as infiltration system using a total storage volume of approximately 15.7 $m^3$. The gravel fill material is separated from the surrounding soil using a geo-textile. There are two inspection chambers, which are built as sediment traps. These are connected with two 150 mm diameter perforated pipes, of which the inflow pipe is plucked at the downstream chamber and the outflow pipe is plucked at the inflow chamber.

### B.5.2 Monitoring information

The infiltration trench at Broxden was monitored for 3½ months from November 2002 until February 2003. Figure B-10 displays a bar chart of the monitoring period at Broxden. Flow data and water quality data was obtained at two monitoring locations, the inflow and outflow of the infiltration trench (see Appendix D.5). Water quality data was monitored using quality sondes providing continuous data and flow data was measured using Sigma 950 flow loggers. Rain data was monitored continuously in two-minute intervals using a Casella raingauge. The raingauge was located approximately 1½ km North of the Broxden housing estate, on premises of Norwich Union.

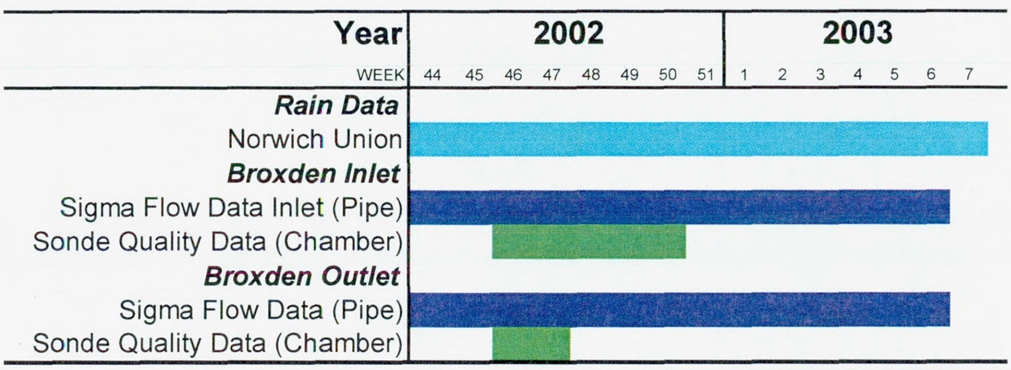



**Figure B-10: Bar chart of monitoring period at Broxden**

16 flow events were recorded at Broxden and peak inflow varies from 12 l/s to 3 l/s producing a total volume of 1.3 and 8.55 mm, respectively. The percentage runoff at this site is 77% on average. The infiltration trench reduces the inflow volume by over 70% with a mean peak flow reduction of almost 80%. Lag time varies from two minutes to just under one hour. Two events were recorded where inflow exceeded 100% and it is thought that this is mainly due to the spatial variation of rainfall. Table B-5 provides a summary of recorded events at Broxden.

| | Rain data | | | Trench Inflow | | | Trench Outflow | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Duration | Total | Peak | Total Flow | Peak Flow | Flow Vol | Total Flow | Peak Flow | Flow Vol | Lag Time | Peak flow reduction |
| | [d:hh:mm] | [mm] | [mm/h] | [mm] | [l/s] | [%] | [mm] | [l/s] | [%] | [hh:mm] | [%] |
| Min | 0:03:08 | 2.4 | 1.5 | 1.3 | 3.1 | 49% | 0.5 | 0.5 | 19 | 00:02 | 59% |
| Max | 1:08:40 | 15.4 | 30.0 | 8.5 | 12.8 | 116% | 2.3 | 3.6 | 41 | 00:56 | 86% |
| Mean | 0:12:26 | 6.1 | 6.1 | 0.8 | 6.5 | 77% | 0.0 | 1.6 | 27 | 00:28 | 77% |

**Table B-5: Summary of events recorded at Broxden**

Outflow monitoring was undertaken at the inlet to the downstream inspection chamber. This location provided good condition for flow monitoring, as it is a straight leg with a relatively low slope. Figure B-11 shows the head-discharge relationship at Broxden and Appendix C.5.2 provides typical hydrographs.

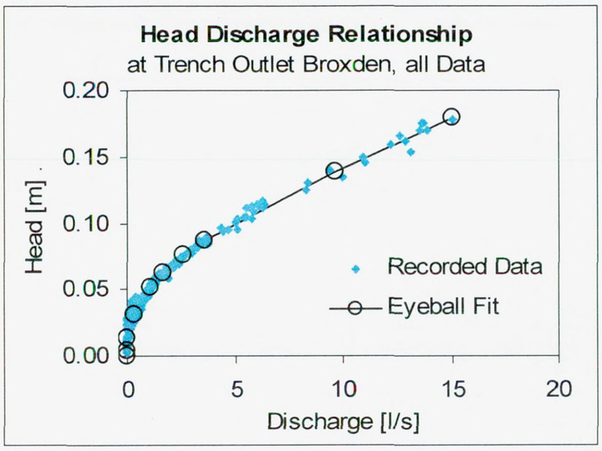



**Figure B-11: Head-discharge relationship, Broxden**

## Appendix - B.6 Infiltration trench at housing estate Transy Estate, Dunfermline

### B.6.1 Site description

The surface water drainage at Transy Estate Dunfermline serves roof and road surfaces of 16 houses on an area of 6568 $m^2$. Road runoff is intercepted via typically trapped gully pots and then conveyed to the end-of-pipe-infiltration trench. Purified storm water is discharged to the local sewerage system. No design calculation is available for this site. Plate B-4 shows a site view of Transy Estate Dunfermline, which was constructed in 1999.

**Plate B-6: Site view of infiltration trench at Transy Estate**

Appendix D.6 shows a detailed construction drawing of the infiltration trench. The system is 28 m long, 10 m wide and 1.5 m deep. It is designed as infiltration system with a total storage volume of approximately 129 $m^3$. The gravel fill material is separated from the surrounding soil using a 1000 Gauge Terram geo-textile or equivalent. There are two inspection chambers, one up- and one downstream to the trench. The inspection upstream to the trench is built as sediment trap and incorporates a 225 mm diameter overflow-bypass discharging to the downstream inspection chamber. The system's outflow is regulated deploying a Hydrobrake prior to discharging to the local sewer. The inflow into the trench is distributed via three legs of 225 mm diameter perforated pipes. The outflow pipe is located approximately 300 mm above the base of the trench and this is disconnected from the inflow pipes The filter material consists of nominal size crushed rock of 100-150 mm.

### B.6.2 Monitoring information

The infiltration trench at Transy was monitored for 6 months from March 2003 until August 2003. Figure B-12 displays a bar chart of the monitoring period at Transy Estate. Flow data was obtained at two monitoring locations, the inflow and outflow of the infiltration trench using Sigma 911 flow loggers. Rain data was monitored continuously in two-minute intervals using a Casella raingauge. The raingauge was located approximately 2.5 km East of the Transy Estate.

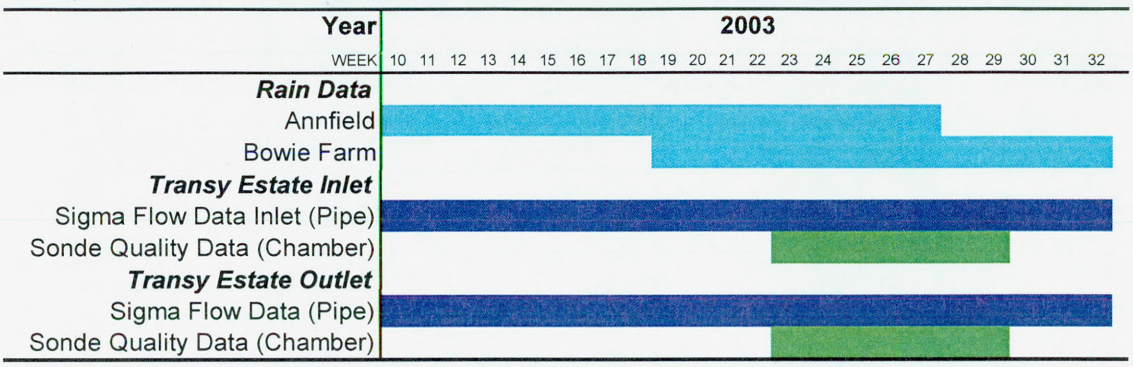



**Figure B-12: Bar chart of monitoring period at Transy**

19 flow events were recorded at Transy and peak inflow varies from 15 l/s to 0.5 l/s producing a total volume of 0.2 and 4.0 mm, respectively. This corresponds to a mean percentage inflow of 16%. The infiltration trench reduces the inflow volume on average by 30% with a mean peak flow reduction of 61%. Only six events were suitable for lag time analyses and data varies from 28 minutes up to 6.5 hours days with an average lag time of 3 hours.

The site was heavily influenced by ground water ingress and this is the reason for the high percentage outflow with frequent outflow of more than 100% and negative peak flow reduction. The extremely low percentage inflow is due to an additional infiltration trench, which is located upstream, receiving runoff from a main part of the drainage area. Table B-6 provides a summary of recorded events.

| | Rain data | | | Trench Inflow | | | Trench Outflow | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Duration | Total | Peak | Total Flow | Peak Flow | Flow Vol | Total Flow | Peak Flow | Flow Vol | Lag Time | Peak flow reduction |
| | [d:hh:mm] | [mm] | [mm/h] | [mm] | [l/s] | [%] | [mm] | [l/s] | [%] | [hh:mm] | [%] |
| Min | 0:01:06 | 2.20 | 0.75 | 0.17 | 0.45 | 3% | 0.00 | 0.00 | 0 | 00:28 | -141% |
| Max | 0:17:14 | 21.40 | 60.00 | 3.84 | 14.87 | 43% | 0.64 | 6.23 | 260 | 06:26 | 100% |
| Mean | 0:06:30 | 5.96 | 10.72 | 0.94 | 3.80 | 16% | 0.15 | 1.12 | 70 | 03:00 | 61% |

**Table B-6: Summary of events recorded at Transy Estate**

Outflow monitoring at Transy was undertaken in the downstream inspection chamber upstream from a Hydrobrake. During the monitoring period the recorded flow was too low to be affected by the Hydrobrake and a good head-discharge relationship was developed and this is shown in Figure B-12. Typical hydrographs from in and outlet are shown in Figure B-13.

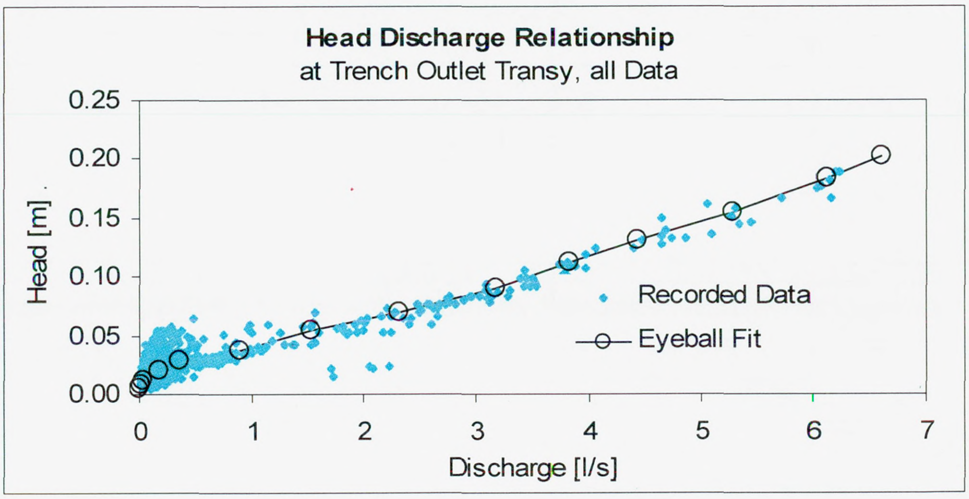



**Figure B-13: Head-discharge relationship, Transy**

# Appendix - C Monitoring Results

## C.1 Monitoring Results, Lang Stracht

*Data recorded from January 2000 until October 2002*

- *C.1.1 List of Events*
- *C.1.2 Typical Hydrographs*
- *C.1.3 Comparison of selected Parameters*

## C.2 Monitoring Results, Spine Road

*Data recorded from March 2003 until June 2003*

- *C.2.1 List of Events*
- *C.2.2 Typical Hydrographs*
- *C.2.3 Comparison of selected Parameters*

## C.3 Monitoring Results, Glencarse

*Data recorded from June 2003 until December 2004*

- *C.3.1 List of Events*
- *C.3.2 Typical Hydrographs*
- *C.3.3 Comparison of selected Parameters*

## C.4 Monitoring Results, Walker Dam

*Data recorded from November 2002 until February 2003*

- *C.4.1 List of Events*
- *C.4.2 Typical Hydrographs*
- *C.4.3 Comparison of selected Parameters*

## C.5 Monitoring Results, Broxden

*Data recorded from November 2002 until February 2003*

- *C.5.1 List of Events*
- *C.5.2 Typical Hydrographs*
- *C.5.3 Comparison of selected Parameters*

## C.6 Monitoring Results, Transy

*Data recorded from March 2003 until August 2003*

- *C.6.1 List of Events*
- *C.6.2 Typical Hydrographs*
- *C.6.3 Comparison of selected Parameters*

# *Appendix - C.1 Monitoring Results, Lang Stracht*

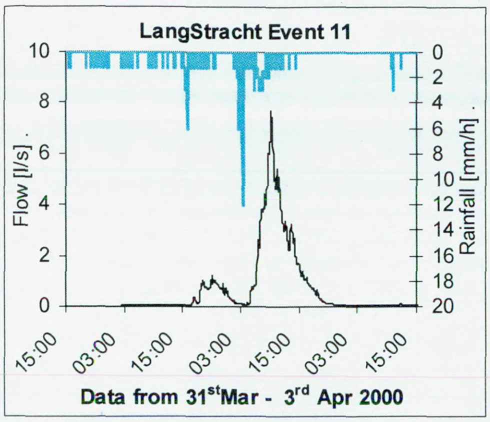
## C.1.1 List of Events

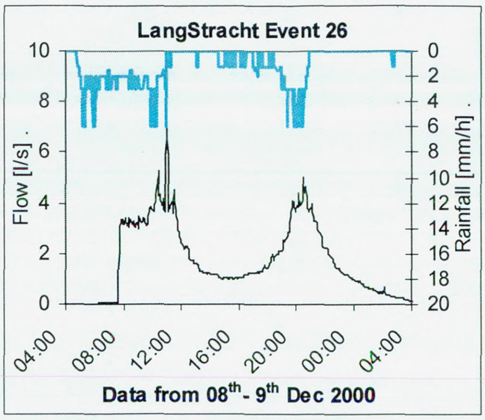
| Event | Rainfall | | | | | Outflow | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Rain Start | | Duration | Total | Peak | Total Flow | Peak Flow | Flow Volume | Lag Time |
| [-] | [dd:hh:mm] | | [d:hh:mm] | [mm] | [mm/h] | [mm] | [l/s] | [%] | [hh:mm] |
| LSA01 | 29/01/00 03:52 | 01/02/00 17:08 | 3:13:16 | 10.8 | 7.2 | 3.0 | 2.5 | 27% | 0:13:55 |
| LSA02 | 06/02/00 19:04 | 08/02/00 13:10 | 1:18:06 | 3.0 | 2 | 0.5 | 0.1 | 18% | 0:12:30 |
| LSA03 | 17/02/00 21:02 | 19/02/00 09:28 | 1:12:26 | 4.0 | 6 | 0.6 | 0.1 | 15% | 0:14:18 |
| LSA04 | 21/02/00 02:24 | 21/02/00 13:30 | 0:11:06 | 2.8 | 3 | 0.4 | 0.1 | 14% | 0:09:30 |
| LSA05 | 03/03/00 03:14 | 04/03/00 11:50 | 1:08:36 | 2.8 | 1.5 | 0.7 | 0.1 | 25% | 0:10:52 |
| LSA06 | 09/03/00 08:56 | 10/03/00 06:36 | 0:21:40 | 3.8 | 6 | 0.7 | 0.3 | 19% | 0:10:30 |
| LSA07 | 23/03/00 18:40 | 26/03/00 02:06 | 2:07:26 | 14.8 | 6 | 10.1 | 11.1 | 68% | 0:02:03 |
| LSA08 | 31/03/00 15:42 | 04/04/00 11:44 | 3:20:02 | 22.6 | 12 | 18.3 | 7.7 | 81% | 0:10:49 |
| LSA09 | 10/04/00 10:22 | 23/04/00 04:47 | 12:18:25 | 16.4 | 8 | 9.8 | 2.0 | 60% | |
| LSA10 | 21/06/00 11:14 | 24/06/00 05:16 | 2:18:02 | 10.0 | 9 | 1.3 | 0.7 | 13% | |
| LSA11 | 20/10/00 11:26 | 20/10/00 18:46 | 0:07:20 | 4.8 | 12 | 0.5 | 0.4 | 10% | 0:05:25 |
| LSA12 | 24/10/00 20:48 | 26/10/00 15:02 | 1:18:14 | 11.8 | 4.5 | 4.2 | 3.3 | 36% | 0:03:09 |
| LSA13 | 27/10/00 06:22 | 27/10/00 13:52 | 0:07:30 | 2.0 | 7.2 | 0.3 | 0.1 | 17% | 0:07:44 |
| LSA14 | 28/10/00 15:06 | 30/10/00 23:12 | 2:08:06 | 14.0 | 7.2 | 6.1 | 7.0 | 44% | |
| LSA15 | 07/12/00 10:36 | 15/12/00 14:48 | 8:04:12 | 53.0 | 12 | 22.5 | 7.7 | 43% | |
| LSA16 | 19/12/00 21:54 | 25/12/00 15:14 | 5:17:20 | 24.8 | 6 | 8.0 | 3.0 | 32% | |
| LSA17 | 04/01/01 04:26 | 05/01/01 19:30 | 1:15:04 | 4.2 | 3 | 0.7 | 0.1 | 16% | 0:17:40 |
| LSA18 | 18/01/01 04:28 | 26/01/01 21:04 | 8:16:36 | 43.0 | 6 | 10.9 | 2.5 | 25% | 0:17:00 |
| LSA19 | 13/04/01 17:28 | 15/04/01 22:34 | 2:05:06 | 3.6 | 1.5 | 1.5 | 0.1 | 41% | 0:09:43 |
| LSA20 | 17/04/01 09:06 | 19/04/01 23:46 | 2:14:40 | 8.6 | 6 | 4.7 | 1.0 | 55% | 0:07:41 |
| LSA21 | 21/04/01 00:18 | 22/04/01 19:36 | 1:19:18 | 3.2 | 2 | 1.4 | 0.7 | 43% | 0:04:23 |
| LSA22 | 27/04/01 23:50 | 30/04/01 12:28 | 2:12:38 | 9.4 | 12 | 4.6 | 2.7 | 49% | 0:04:07 |
| LSA23 | 16/05/01 01:08 | 16/05/01 06:48 | 0:05:40 | 2.2 | 2 | 0.1 | 0.1 | 4% | |
| LSA24 | 17/05/01 05:12 | 17/05/01 11:02 | 0:05:50 | 6.0 | 6 | 1.9 | 1.4 | 32% | |
| LSA25 | 15/06/01 02:58 | 16/06/01 06:00 | 1:03:02 | 4.2 | 1.5 | 1.1 | 6.9 | 26% | 0:04:10 |
| LSA26 | 19/06/01 04:24 | 19/06/01 21:10 | 0:16:46 | 4.2 | 3 | 0.9 | 0.8 | 22% | 0:04:34 |
| LSA27 | 28/06/01 05:20 | 29/06/01 21:02 | 1:15:42 | 3.4 | 3 | 0.5 | 0.6 | 15% | 0:06:55 |
| LSA28 | 06/07/01 18:40 | 07/07/01 08:40 | 0:14:00 | 3.0 | 3 | 0.5 | 0.4 | 18% | 0:09:08 |
| LSA29 | 10/07/01 14:52 | 12/07/01 21:36 | 2:06:44 | 16.2 | 9 | 3.6 | 1.2 | 22% | 0:08:37 |
| LSA30 | 13/07/01 17:04 | 14/07/01 01:24 | 0:08:20 | 6.6 | 12 | 1.7 | 1.5 | 26% | 0:04:10 |
| LSA31 | 10/08/01 09:16 | 11/08/01 00:42 | 0:15:26 | 3.4 | 7.2 | 0.7 | 0.3 | 20% | 0:11:09 |
| LSA32 | 11/08/01 12:08 | 11/08/01 16:48 | 0:04:40 | 4.4 | 3 | 2.1 | 1.4 | 47% | 0:05:19 |
| LSA33 | 15/08/01 22:28 | 17/08/01 03:30 | 1:05:02 | 21.4 | 12 | 8.5 | 3.5 | 40% | 0:03:59 |
| LSA34 | 30/08/01 09:20 | 31/08/01 06:50 | 0:21:30 | 7.6 | 6 | 2.6 | 1.9 | 34% | 0:04:32 |
| LSA35 | 01/09/01 15:28 | 02/09/01 01:16 | 0:09:48 | 4.0 | 12 | 0.9 | 0.5 | 23% | 0:07:37 |
| LSA36 | 07/09/01 06:58 | 09/09/01 21:20 | 2:14:22 | 17.6 | 12 | 5.7 | 2.6 | 32% | |
| LSA37 | 12/09/01 09:40 | 13/09/01 11:06 | 1:01:26 | 4.6 | 3 | 0.9 | 0.3 | 19% | 0:09:29 |
| LSA38 | 16/09/01 03:50 | 16/09/01 12:56 | 0:09:06 | 3.2 | 2 | 0.8 | 0.5 | 26% | 0:05:22 |
| LSA39 | 20/09/01 07:38 | 20/09/01 21:22 | 0:13:44 | 9.6 | 6 | 5.2 | 2.7 | 54% | 0:04:48 |
| LSA40 | 05/10/01 23:22 | 06/10/01 05:06 | 0:05:44 | 8.0 | 12 | 2.8 | 2.0 | 35% | 0:04:46 |
| LSA41 | 07/10/01 21:30 | 11/10/01 06:10 | 3:08:40 | 32.2 | 12 | 12.7 | 4.5 | 39% | 0:06:26 |
| LSA42 | 15/10/01 01:00 | 15/10/01 12:34 | 0:11:34 | 2.4 | 6 | 0.5 | 0.4 | 22% | 0:05:49 |
| LSA43 | 18/10/01 04:48 | 18/10/01 23:50 | 0:19:02 | 2.0 | 1.2 | 0.2 | 0.1 | 9% | 0:09:53 |
| LSA44 | 30/10/01 02:36 | 31/10/01 12:20 | 1:09:44 | 6.0 | 12 | 1.1 | 0.5 | 19% | 0:06:47 |
| LSA45 | 04/11/01 23:54 | 06/11/01 11:18 | 1:11:24 | 5.2 | 6 | 1.0 | 0.5 | 20% | 0:09:39 |
| LSA46 | 07/11/01 23:52 | 14/11/01 20:08 | 6:20:16 | 20.4 | 6 | 6.8 | 0.7 | 33% | 0:09:55 |
| LSA47 | 22/11/01 05:28 | 22/11/01 09:08 | 0:03:40 | 3.6 | 6 | 0.8 | 0.5 | 23% | 0:05:36 |
| LSA48 | 23/11/01 08:24 | 23/11/01 18:12 | 0:09:48 | 2.0 | 2 | 0.2 | 0.1 | 11% | 0:10:22 |
| LSA49 | 30/11/01 01:36 | 01/12/01 03:24 | 1:01:48 | 7.2 | 6 | 2.1 | 1.0 | 29% | 0:06:53 |
| LSA50 | 03/12/01 16:40 | 05/12/01 11:24 | 1:18:44 | 24.8 | 12 | 6.7 | 3.0 | 27% | 0:05:38 |
| LSA51 | 20/01/02 09:20 | 21/01/02 09:14 | 0:23:54 | 6.4 | 6 | 1.7 | 0.8 | 26% | 0:05:51 |
| LSA52 | 23/01/02 08:44 | 08/02/02 21:40 | 16:12:56 | 61.8 | 12 | 12.2 | 1.3 | 20% | |
| LSA53 | 10/02/02 23:40 | 12/02/02 08:20 | 1:08:40 | 4.2 | 9 | 0.4 | 0.2 | 10% | 0:09:54 |
| LSA54 | 18/02/02 19:24 | 22/02/02 07:12 | 3:11:48 | 15.2 | 12 | 3.2 | 1.4 | 21% | 0:05:20 |
| LSA55 | 24/02/02 10:12 | 02/03/02 20:02 | 6:09:50 | 16.0 | 7.2 | 3.4 | 0.9 | 21% | 0:01:12 |
| LSA56 | 06/03/02 06:04 | 07/03/02 13:50 | 1:07:46 | 2.0 | 6 | 0.1 | 0.1 | 7% | 0:12:05 |
| LSA57 | 08/03/02 16:26 | 08/03/02 17:34 | 0:01:08 | 2.4 | 6 | 0.1 | 0.1 | 6% | 0:06:15 |
| LSA58 | 10/03/02 07:16 | 10/03/02 20:46 | 0:13:30 | 8.2 | 9 | 1.3 | 0.6 | 16% | 0:07:29 |
| LSA59 | 16/03/02 02:24 | 16/03/02 11:14 | 0:08:50 | 2.4 | 1.2 | 0.4 | 0.3 | 18% | 0:06:36 |
| LSA60 | 21/03/02 02:42 | 23/03/02 09:20 | 2:06:38 | 8.0 | 2 | 2.0 | 0.6 | 25% | 0:04:09 |

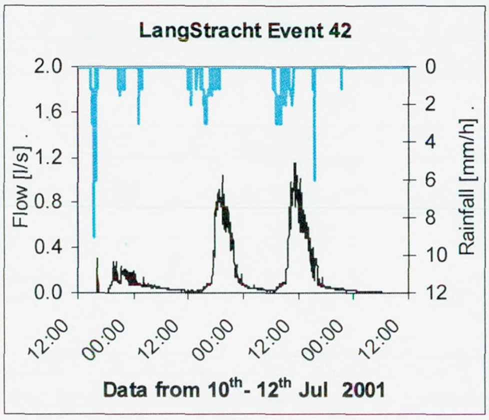
## List of Events continued, Lang Stracht

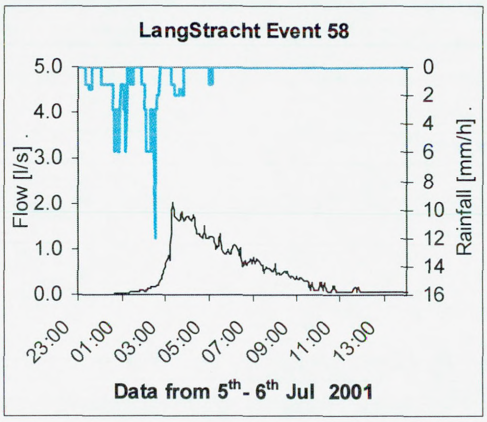
| Event [-] | Rainfall Rain Start [dd:hh:mm] | | Duration [d:hh:mm] | Total [mm] | Peak [mm/h] | Outflow Total Flow [mm] | Peak Flow [l/s] | Flow Volume [%] | Lag Time [hh:mm] |
|---|---|---|---|---|---|---|---|---|---|
| LSA61 | 21/03/02 02:42 | 23/03/02 09:20 | 2:06:38 | 5.2 | 3 | 1.1 | 0.7 | 21% | 0:05:04 |
| LSA62 | 01/04/02 14:44 | 01/04/02 22:24 | 0:07:40 | 2.0 | 8 | 0.3 | 0.2 | 14% | 0:08:31 |
| LSA63 | 02/04/02 21:16 | 03/04/02 02:38 | 0:05:22 | 5.6 | 6 | 1.2 | 0.7 | 21% | 0:06:12 |
| LSA64 | 11/04/02 13:24 | 11/04/02 22:50 | 0:09:26 | 2.4 | 6 | 0.3 | 0.2 | 11% | 0:08:08 |
| LSA65 | 14/04/02 11:28 | 14/04/02 20:38 | 0:09:10 | 4.6 | 3 | 1.6 | 0.5 | 34% | 0:10:33 |
| LSA66 | 28/04/02 12:12 | 29/04/02 14:10 | 1:01:58 | 5.2 | 6 | 1.7 | 0.8 | 33% | 0:06:08 |
| LSA67 | 04/05/02 00:40 | 05/05/02 05:44 | 1:05:04 | 10.2 | 9 | 7.0 | 2.0 | 68% | 0:07:20 |
| LSA68 | 13/05/02 02:58 | 14/05/02 00:20 | 0:21:22 | 4.0 | 3 | 1.9 | 0.9 | 47% | 0:07:34 |
| LSA69 | 15/05/02 23:54 | 16/05/02 16:24 | 0:16:30 | 3.8 | 1.5 | 1.5 | 0.8 | 40% | 0:09:12 |
| LSA70 | 18/05/02 13:00 | 19/05/02 01:08 | 0:12:08 | 14.8 | 9 | 7.7 | 4.5 | 52% | 0:05:48 |
| LSA71 | 21/05/02 00:36 | 22/05/02 22:38 | 1:22:02 | 4.4 | 6 | 1.7 | 0.8 | 39% | 0:06:18 |
| LSA72 | 24/05/02 05:46 | 24/05/02 17:50 | 0:12:04 | 2.6 | 12 | 0.7 | 0.4 | 25% | 0:08:29 |
| LSA73 | 26/05/02 19:56 | 28/05/02 03:38 | 1:07:42 | 5.2 | 6 | 1.3 | 0.7 | 25% | 0:06:24 |
| LSA74 | 02/06/02 05:22 | 03/06/02 13:42 | 1:08:20 | 12.8 | 12 | 4.7 | 4.5 | 37% | 0:04:14 |
| LSA75 | 05/06/02 09:49 | 05/06/02 17:36 | 0:07:47 | 16.8 | 12 | 6.6 | 4.7 | 39% | 0:04:25 |
| LSA76 | 08/06/02 03:30 | 10/06/02 08:06 | 2:04:36 | 2.6 | 6 | 0.4 | 0.1 | 14% | 0:04:31 |
| LSA77 | 25/06/02 23:16 | 26/06/02 18:24 | 0:19:08 | 3.2 | 3 | 0.6 | 0.3 | 18% | 0:11:36 |
| LSA78 | 27/06/02 05:58 | 28/06/02 03:18 | 0:21:20 | 25.6 | 12 | 6.4 | 1.3 | 25% | 0:09:14 |
| LSA79 | 29/06/02 09:48 | 03/07/02 05:02 | 3:19:14 | 5.6 | 3 | 1.5 | 0.9 | 26% | 0:07:29 |
| LSA80 | 07/07/02 13:38 | 08/07/02 06:42 | 0:17:04 | 15.6 | 6 | 8.7 | 3.4 | 56% | 0:05:22 |
| LSA81 | 15/07/02 05:00 | 17/07/02 12:32 | 2:07:32 | 53.0 | 12 | 25.3 | 4.3 | 48% | 0:01:10 |
| LSA82 | 19/07/02 06:32 | 25/07/02 14:34 | 6:08:02 | 20.8 | 3 | 9.2 | 2.1 | 44% | |
| LSA83 | 28/07/02 14:24 | 30/07/02 14:16 | 1:23:52 | 6.0 | 2.4 | 2.9 | 1.3 | 48% | 0:05:00 |
| LSA84 | 23/08/02 14:52 | 24/08/02 03:58 | 0:13:06 | 12.8 | 6 | 5.8 | 2.2 | 45% | 0:03:29 |
| LSA85 | 30/08/02 06:58 | 30/08/02 23:58 | 0:17:00 | 2.2 | 4 | 0.3 | 0.2 | 15% | 0:08:21 |
| LSA86 | 07/09/02 06:22 | 07/09/02 14:58 | 0:08:36 | 4.0 | 12 | 1.3 | 0.8 | 32% | 0:04:36 |
| LSA87 | 08/09/02 20:28 | 09/09/02 00:58 | 0:04:30 | 3.0 | 6 | 0.5 | 0.2 | 17% | 0:08:13 |
| | **29/01/00 03:52** | | **00: 01:08** | **2.0** | **1.2** | **0.1** | **0.1** | **4%** | **01:10** |
| | **08/09/02 20:28** | | **16: 12:56** | **61.8** | **12.0** | **25.3** | **11.1** | **81%** | **17:40** |
| | **16/09/01 12:21** | | **01: 22:54** | **10.2** | **6.6** | **3.6** | **1.6** | **29%** | **07:20** |

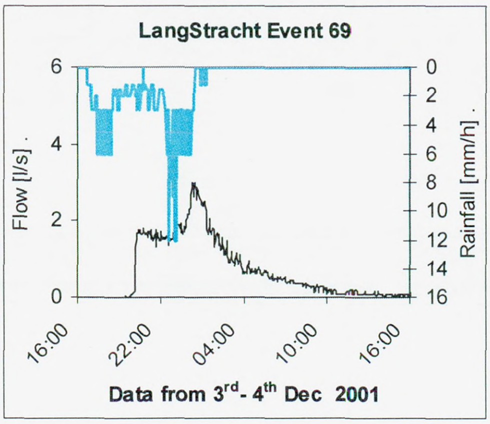
## C.1.2 Typical Hydrographs, Lang Stracht,

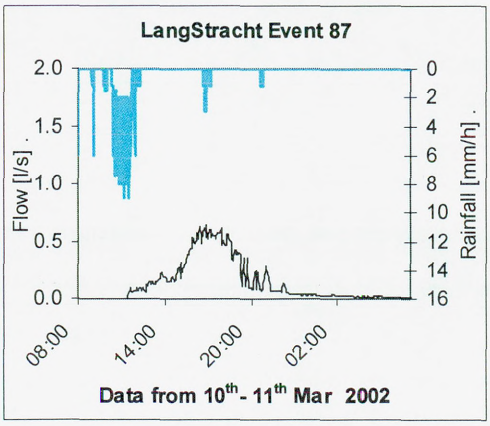


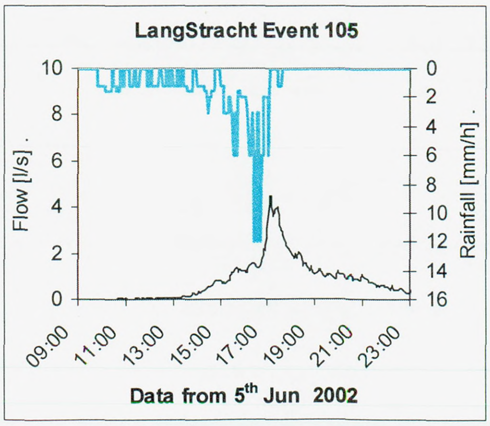



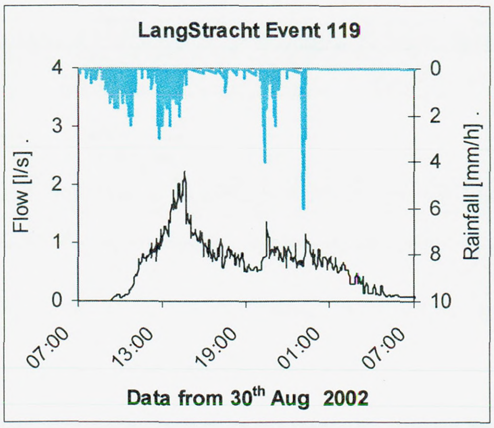



## C.1.3 Comparison of selected Parameters, Lang Stracht

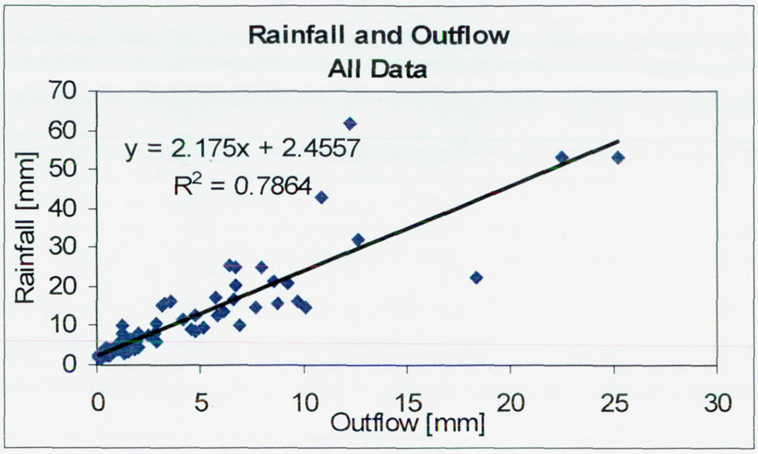



**Figure C-1: Rainfall compared with Flow**

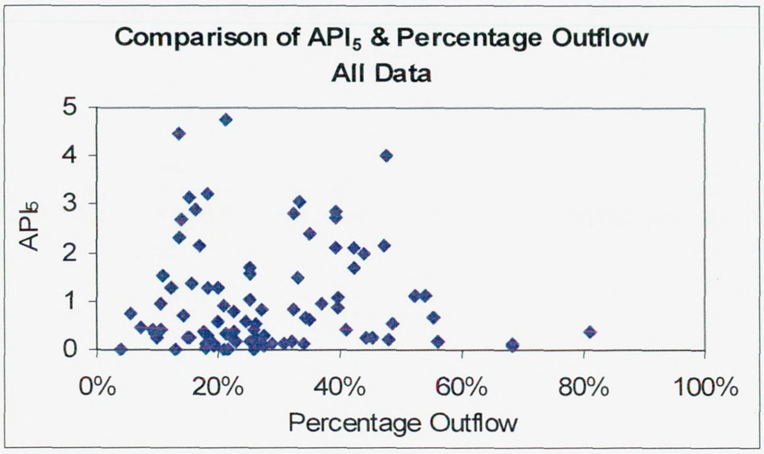



**Figure C-2: $API_5$ compared with Percentage Outflow**

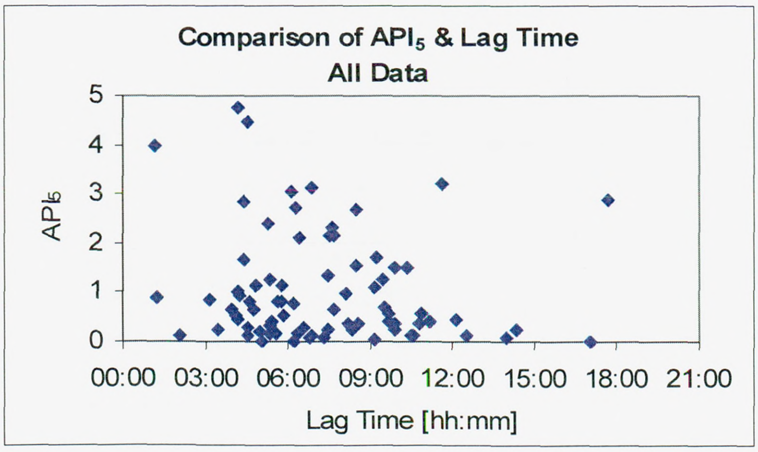


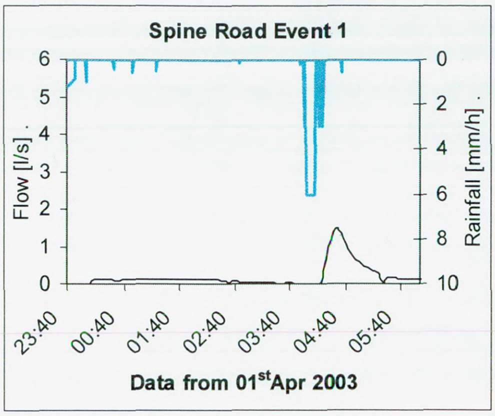

**Figure C-3: $API_5$ compared with Lag Time**

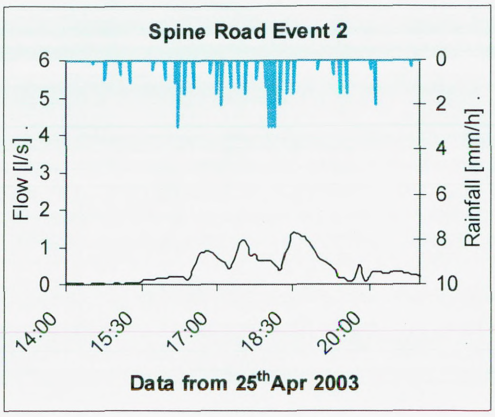
# *Appendix - C.2 Monitoring Results, Spine Road*

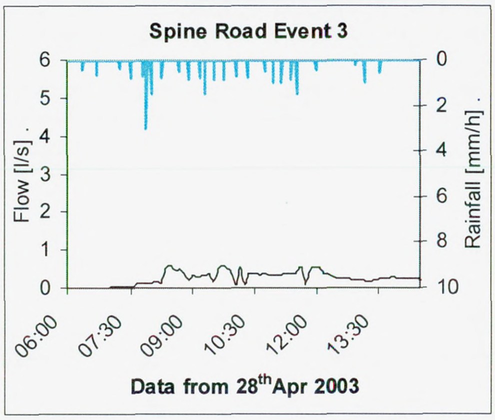
## C.2.1 List of Events

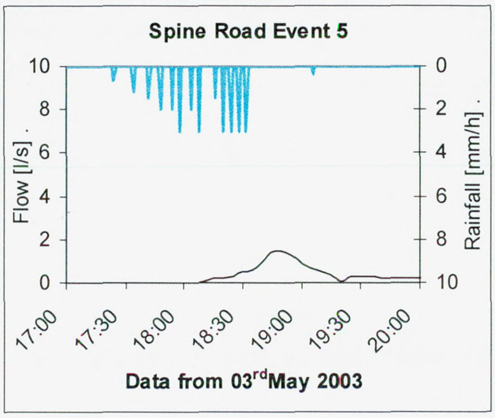
| Event Number [-] | Rainfall: Rain Start [dd:hh:mm] | Rainfall: Duration [d:hh:mm] | Rainfall: Total Rain [mm] | Rainfall: Peak Rain [mm/h] | Upstream: Total Flow [mm] | Upstream: Peak Flow [l/s] | Upstream: Flow Vol [%] | Downstream: Total Flow [mm] | Downstream: Peak Flow [l/s] | Downstream: Flow VoL. [%] | Peak flow increase [%] | Lag Time [hh:mm] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| SP1 | 31/03/03 23:30 | 0:04:50 | 3.3 | 6.0 | 0.00 | 0.00 | 0% | 1.21 | 1.49 | 37% | | |
| SP2 | 25/04/03 14:32 | 0:06:18 | 6.0 | 3.0 | 1.40 | 0.49 | 23% | 3.93 | 1.38 | 66% | 182% | 00:27 |
| SP3 | 28/04/03 04:56 | 1:04:06 | 9.4 | 6.0 | 1.71 | 1.13 | 18% | 2.88 | 2.12 | 31% | 88% | |
| SP4 | 30/04/03 02:48 | 2:23:44 | 37.6 | 24.0 | 15.21 | 5.13 | 40% | 31.62 | 18.20 | 84% | 255% | |
| SP5 | 03/05/03 17:06 | 1:02:44 | 3.4 | 3.0 | 0.46 | 0.59 | 14% | 1.59 | 1.44 | 47% | 144% | 01:46 |
| SP6 | 04/05/03 20:16 | 0:22:42 | 4.6 | 3.0 | 0.73 | 0.55 | 16% | 2.72 | 1.24 | 59% | 125% | 01:09 |
| SP7 | 10/05/03 22:08 | 0:03:18 | 3.0 | 6.0 | 1.93 | 2.95 | 64% | 3.31 | 4.72 | 110% | 60% | 00:40 |
| SP8 | 11/05/03 19:40 | 1:08:38 | 9.2 | 12.0 | 1.42 | 1.45 | 15% | 3.24 | 2.68 | 35% | 85% | 03:31 |
| SP9 | 14/05/03 14:32 | 0:04:28 | 5.8 | 12.0 | 2.02 | 0.96 | 35% | 4.26 | 1.96 | 74% | 104% | 00:28 |
| SP10 | 16/05/03 09:51 | 1:05:20 | 10.8 | 6.0 | 1.70 | 0.59 | 16% | 7.28 | 1.44 | 67% | 144% | 02:02 |
| SP11 | 18/05/03 11:12 | 0:08:28 | 7.6 | 30.0 | 2.12 | 3.53 | 28% | 4.69 | 4.45 | 62% | 26% | 00:53 |
| **Min** | | **0:03:18** | **3.0** | **3.0** | **0.46** | **0.49** | **14%** | **1.59** | **1.24** | **31%** | **26%** | **00:27** |
| **Max** | | **2:23:44** | **37.6** | **30.0** | **15.21** | **5.13** | **64%** | **31.62** | **18.20** | **110%** | **255%** | **03:31** |
| **Mean** | | **0:23:22** | **9.7** | **10.5** | **2.87** | **1.74** | **27%** | **6.55** | **3.96** | **63%** | **121%** | **01:22** |

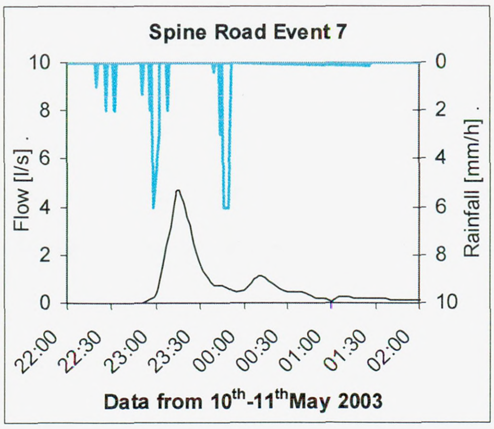
## C.2.2 Typical Hydrographs, Spine Road

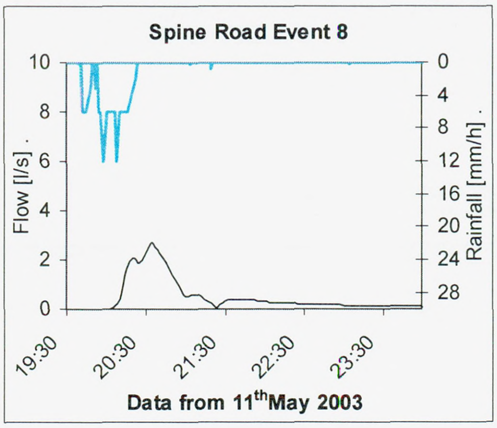


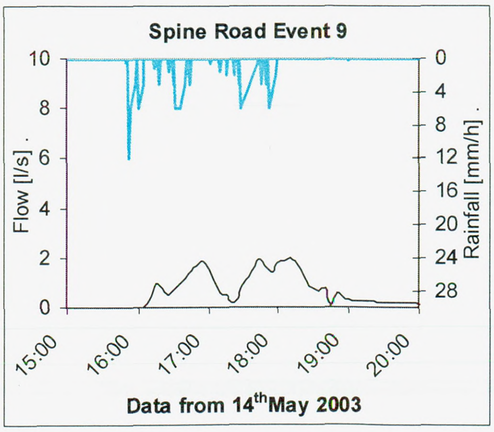



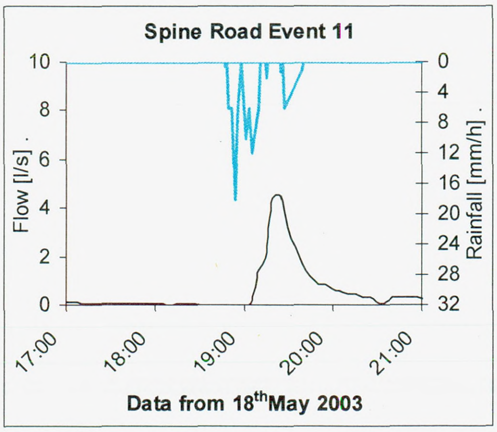



### C.2.3 Comparison of selected Parameters, Spine Road

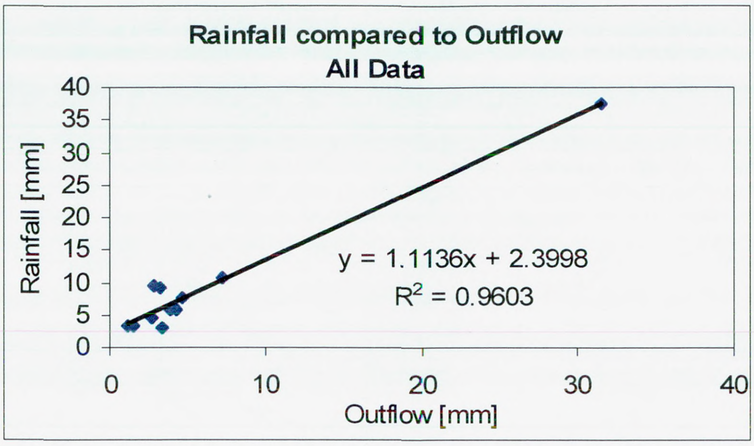



**Figure C-4: Rainfall compared with Flow**

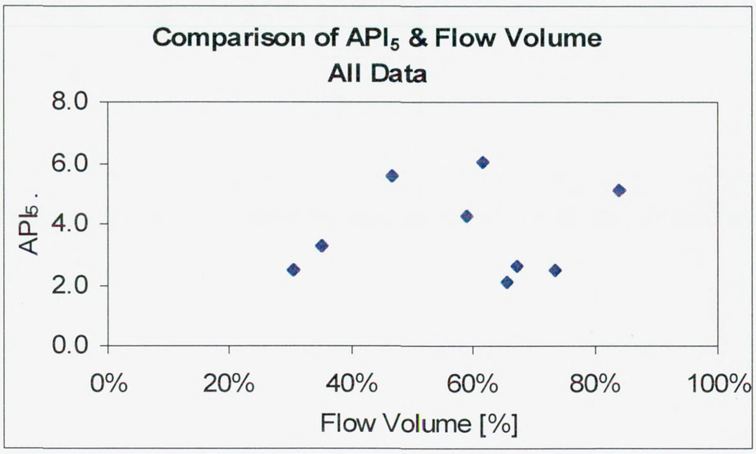



**Figure C-5: $API_5$ compared with Percentage Flow Volume**

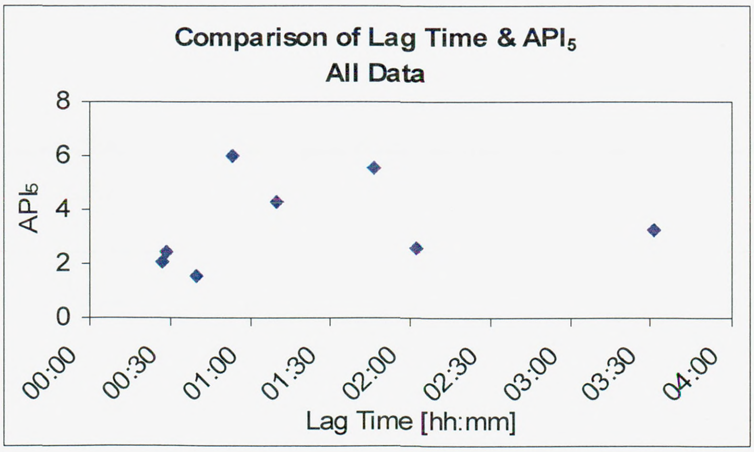


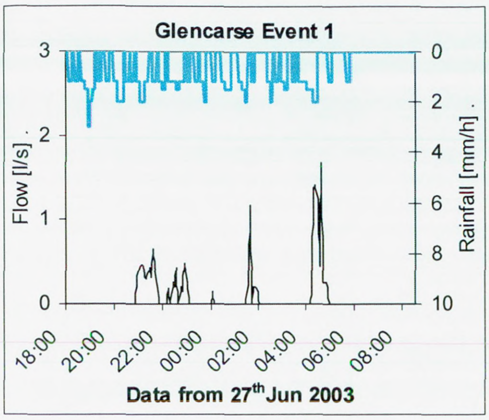

**Figure C-6: $API_5$ compared with Lag Time**

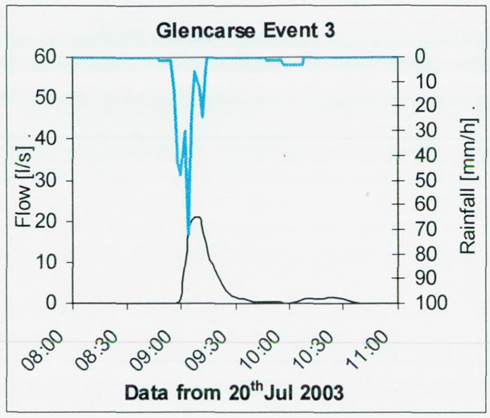
# *Appendix - C.3 Monitoring Results, Glencarse*

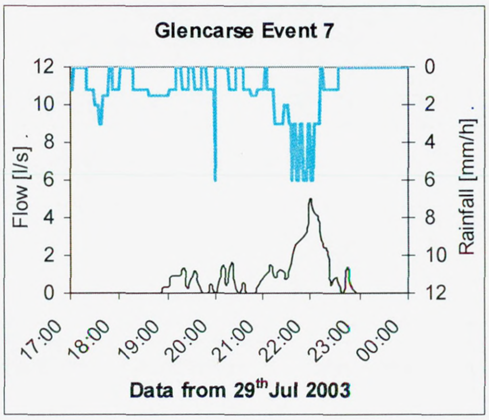
## C.3.1 List of Events

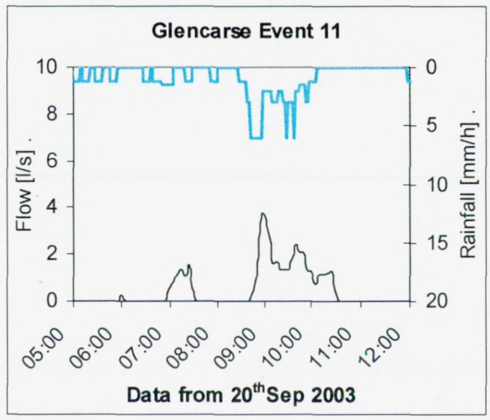
| | Rainfall | | | | Upstream Monitoring Location | | | |
|---|---|---|---|---|---|---|---|---|
| Event Number [-] | Rain Start [dd:hh:mm] | Duration of Rainfall [d:hh:mm] | Total Rain [mm] | Peak Rainfall [mm/h] | Total Flow [mm] | Peak Flow [l/s] | Flow Volume [%] | Lag Time [hh:mm] |
| GC1 | 27/06/2003 18:05 | 0:16:34 | 10.8 | 3.0 | 1.77 | 1.67 | 16% | 01:28 |
| GC2 | 01/07/2003 06:13 | 0:01:56 | 2.6 | 6.0 | 0.91 | 1.97 | 35% | 00:33 |
| GC3 | 20/07/2003 08:47 | 0:01:22 | 10.2 | 72.0 | 8.60 | 20.94 | 84% | 00:09 |
| GC4 | 21/07/2003 12:17 | 0:00:40 | 2.0 | 6.0 | 0.77 | 1.49 | 39% | 00:26 |
| GC5 | 22/07/2003 03:17 | 0:03:50 | 6.4 | 6.0 | 5.84 | 2.98 | 91% | 00:37 |
| GC6 | 28/07/2003 14:59 | 0:05:56 | 6.0 | 6.0 | 1.90 | 1.84 | 32% | 00:39 |
| GC7 | 29/07/2003 15:41 | 0:06:54 | 8.6 | 6.0 | 6.32 | 4.99 | 73% | 01:05 |
| GC8 | 30/07/2003 15:31 | 0:01:12 | 2.4 | 6.0 | 0.76 | 2.09 | 32% | 00:30 |
| GC9 | 18/08/2003 01:35 | 0:01:18 | 2.2 | 6.0 | 0.35 | 0.92 | 16% | 00:28 |
| GC10 | 07/09/2003 19:45 | 0:05:10 | 5.0 | 12.0 | 2.10 | 2.77 | 42% | 01:18 |
| GC11 | 20/09/2003 02:27 | 0:07:40 | 8.2 | 6.0 | 4.95 | 3.75 | 60% | 01:32 |
| GC12 | 21/09/2003 21:09 | 0:07:30 | 12.4 | 6.0 | 11.78 | 4.06 | 95% | 00:47 |
| GC13 | 22/10/2003 05:50 | 0:01:20 | 2.8 | 6.0 | 0.78 | 1.44 | 28% | 00:35 |
| GC14 | 22/10/2003 22:26 | 0:00:14 | 2.0 | 36.0 | 1.51 | 3.43 | 75% | 00:16 |
| GC15 | 01/11/2003 22:12 | 0:03:18 | 2.8 | 3.0 | 0.20 | 0.46 | 7% | 01:43 |
| GC16 | 09/11/2003 04:54 | 0:01:40 | 2.2 | 12.0 | 0.27 | 0.76 | 12% | 00:38 |
| GC17 | 25/11/2003 23:56 | 0:03:16 | 2.8 | 2.0 | 1.12 | 1.50 | 40% | 00:52 |
| GC18 | 28/11/2003 13:56 | 0:02:38 | 2.6 | 6.0 | 1.33 | 1.77 | 51% | 00:41 |
| GC19 | 29/11/2003 09:38 | 0:09:18 | 13.0 | 4.0 | 9.19 | 2.38 | 71% | 00:00 |
| **Min** | | **0:00:14** | **2.0** | **2.0** | **0.20** | **0.46** | **7%** | **00:09** |
| **Max** | | **0:16:34** | **13.0** | **72.0** | **11.78** | **20.94** | **95%** | **01:43** |
| **Mean** | | **0:04:18** | **5.5** | **11.1** | **3.18** | **3.22** | **47%** | **00:48** |

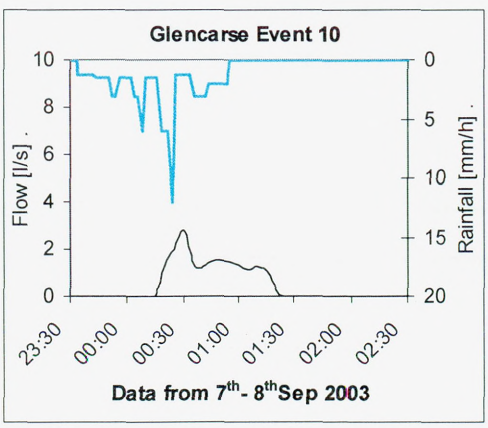
## C.3.2 Typical Hydrographs, Glencarse

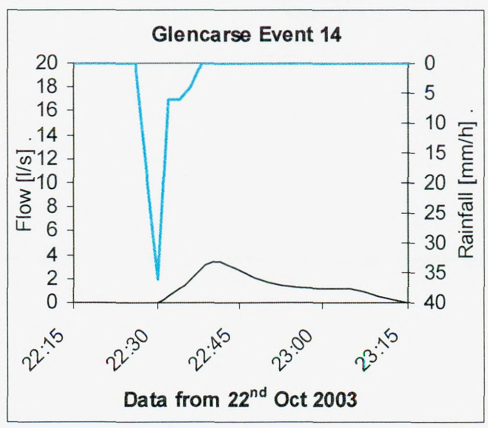


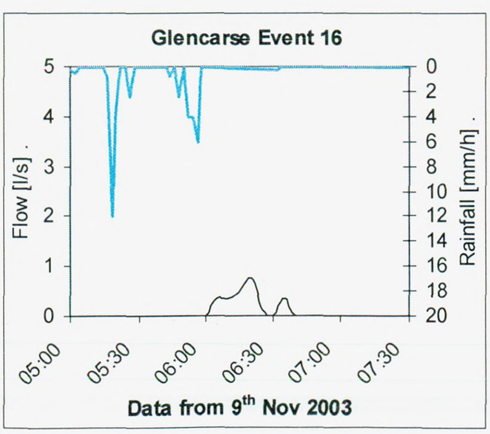



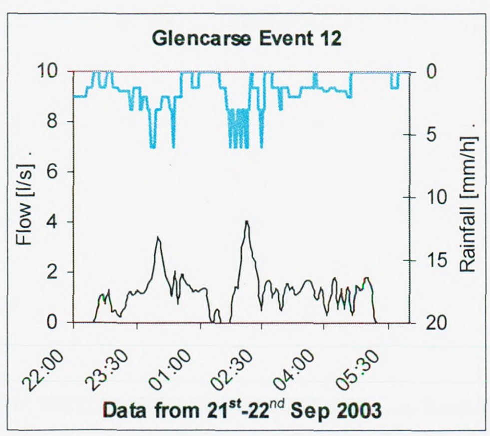



### C.3.3 Comparison of selected Parameters, Glencarse

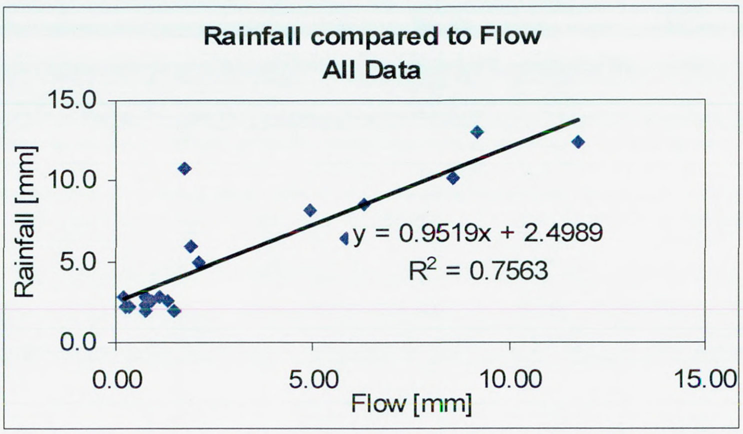



**Figure C-7: Rainfall compared with Flow**

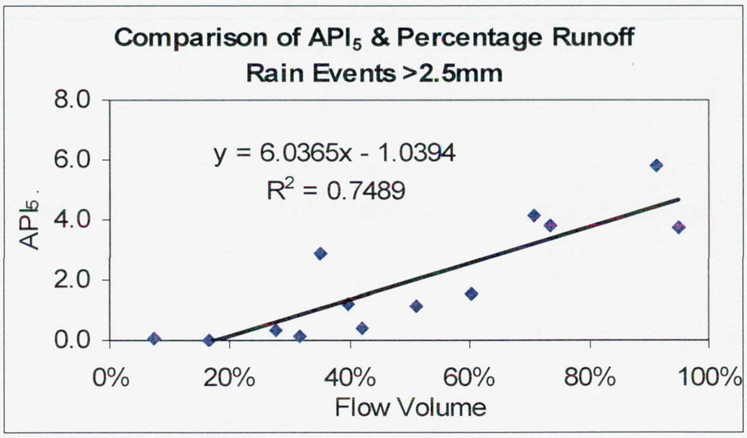



**Figure C-8: $API_5$ compared with Percentage Flow Volume**

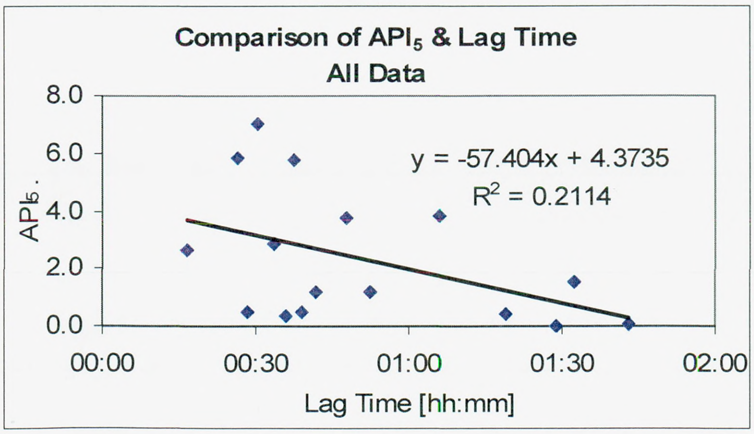


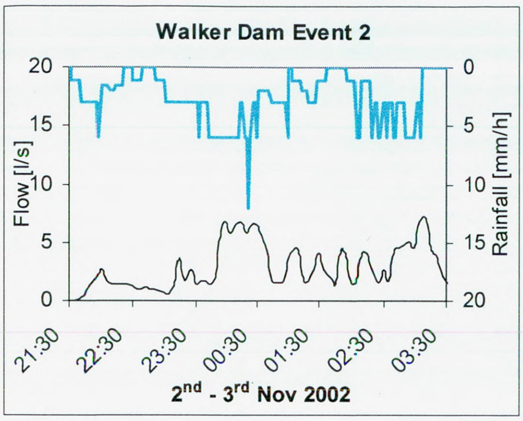

**Figure C-9: $API_5$ compared with Lag Time**

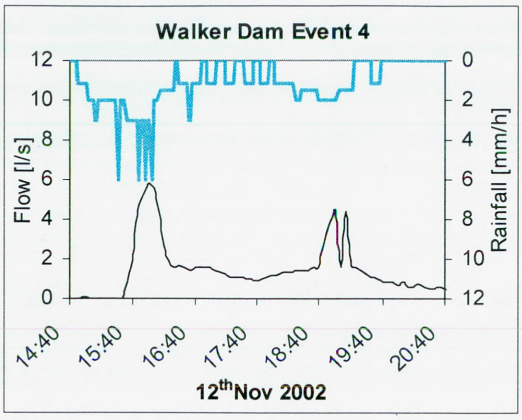
# *Appendix - C.4 Monitoring Results, Walker Dam*

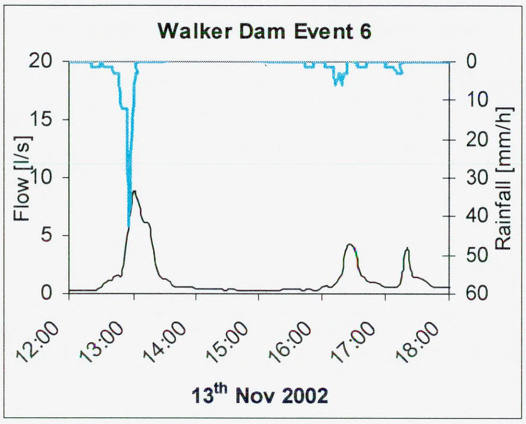
## C.4.1 List of Events

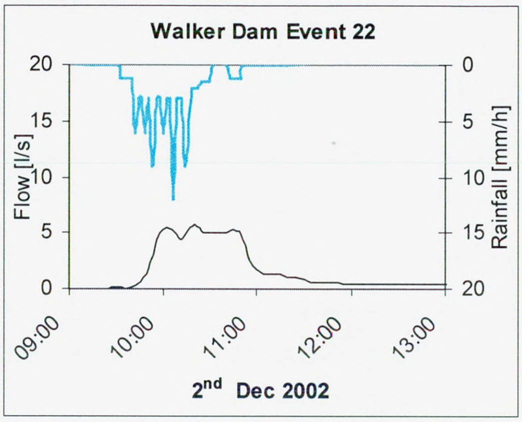
| | Rainfall | | | | Trench Inflow | | | Trench Outflow | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Event** | **Rain Start** | **Duration** | **Total** | **Peak** | **Total Flow** | **Peak Flow** | **Flow Vol.** | **Total Flow** | **Peak Flow** | **Flow Vol.** | **Lag Time**** | **Peak flow reduction** |
| [-] | [dd:hh:mm] | [d:hh:mm] | [mm] | [mm/h] | [mm] | [l/s] | [%] | [mm] | [l/s] | [%] | [hh:mm] | [%] |
| WD1 | 01/11/02 11:05 | 0:05:32 | 2.8 | 6.0 | 1.27 | 4.76 | 45% | 1.48 | 1.38 | 117% | 00:20 | 71% |
| WD2 | 02/11/02 21:31 | 0:09:56 | 16.2 | 12.0 | 12.73 | 7.62 | 79% | 10.04 | 7.14 | 79% | 00:21 | 6% |
| WD3 | 10/11/02 06:59 | 0:04:10 | 5.4 | 3.0 | 5.36 | 8.68 | 99% | 2.51 | 3.34 | 47% | 00:41 | 61% |
| WD4 | 12/11/02 14:47 | 0:04:52 | 7.4 | 6.0 | 6.91 | 8.91 | 93% | 4.59 | 5.80 | 66% | 00:11 | 35% |
| WD5 | 13/11/02 06:27 | 0:04:50 | 6.4 | 36.0 | 2.28 | 11.54 | 36% | 2.02 | 8.41 | 89% | 00:09 | 27% |
| WD6 | 13/11/02 12:19 | 0:04:58 | 7.0 | 42.0 | 3.47 | 13.88 | 50% | 3.80 | 8.74 | 110% | 00:00 | 37% |
| WD7 | 14/11/02 09:19 | 0:08:34 | 9.4 | 3.0 | 5.37 | 4.07 | 57% | 4.18 | 2.75 | 78% | 00:35 | 32% |
| WD8 | 14/11/02 21:57 | 0:08:04 | 10.4 | 12.0 | 7.90 | 5.78 | 76% | 5.31 | 5.37 | 67% | | 7% |
| WD9 | 20/11/02 03:49 | 0:01:38 | 3.6 | 30.0 | 1.79 | 8.39 | 50% | 1.27 | 3.04 | | 00:25 | 64% |
| WD10 | 20/11/02 08:23 | 0:03:22 | 5.2 | 6.0 | 3.33 | 3.90 | 64% | 3.76 | 4.51 | 113% | | -16% |
| WD11 | 20/11/02 12:55 | 0:05:46 | 12.2 | 6.0 | 9.60 | 5.40 | 79% | 9.36 | 5.98 | 98% | 00:16 | -11% |
| WD12 | 20/11/02 22:09 | 0:04:16 | 4.4 | 12.0 | 3.69 | 4.77 | 84% | 3.51 | 4.51 | 95% | 00:05 | 5% |
| WD13 | 21/11/02 05:47 | 0:02:10 | 2.2 | 3.0 | 1.58 | 2.97 | 72% | 1.25 | 1.61 | 79% | 00:06 | 46% |
| WD14 | 21/11/02 19:13 | 0:06:04 | 4.4 | 3.0 | 4.70 | 1.95 | 107% | 3.23 | 1.12 | 69% | | 43% |
| WD15 | 23/11/02 01:00 | 0:01:22 | 5.2 | 30.0 | 1.41 | 5.54 | 27% | 1.98 | 5.80 | 0% | 00:28 | -5% |
| WD16 | 23/11/02 06:46 | 0:02:34 | 2.6 | 6.0 | 2.52 | 3.95 | 97% | 1.43 | 1.20 | 57% | 00:18 | 70% |
| WD17 | 23/11/02 19:46 | 0:03:04 | 5.4 | 12.0 | 2.86 | 5.44 | 53% | 1.44 | 4.10 | 50% | 00:20 | 25% |
| WD18 | 27/11/02 04:54 | 0:04:04 | 2.0 | 1.2 | 1.37 | 2.86 | 69% | 0.70 | 0.81 | 51% | 00:19 | 72% |
| WD19 | 27/11/02 19:36 | 0:05:02 | 9.6 | 6.0 | 7.98 | 4.93 | 83% | 3.46 | 2.49 | 43% | | 49% |
| WD20 | 30/11/02 14:26 | 0:07:00 | 4.4 | 3.0 | 2.35 | 2.71 | 53% | 1.58 | 0.85 | 67% | 00:34 | 69% |
| WD21 | 01/12/02 10:16 | 0:03:16 | 9.0 | 9.0 | 6.78 | 5.79 | 75% | 2.32 | 2.75 | 34% | 00:14 | 52% |
| WD22 | 02/12/02 03:00 | 1:13:04 | 16.0 | 15.0 | 14.38 | 7.62 | 90% | 15.84 | 5.98 | 110% | 00:40 | 22% |
| WD23 | 04/12/02 01:34 | 0:13:18 | 12.0 | 9.0 | 11.72 | 4.54 | 98% | 6.63 | 2.24 | 57% | | 51% |
| WD24 | 05/12/02 02:58 | 0:07:14 | 7.8 | 12.0 | 7.46 | 8.29 | 96% | 9.18 | 6.48 | 123% | | 22% |
| WD25 | 15/12/02 14:36 | 0:05:04 | 4.4 | 12.0 | 2.47 | 5.00 | 56% | 1.04 | 1.12 | 42% | 00:45 | 78% |
| WD26 | 16/12/02 10:50 | 0:06:42 | 3.0 | 6.0 | 2.40 | 4.71 | 80% | 1.29 | 1.79 | 54% | 00:19 | 62% |
| WD27 | 21/12/02 09:40 | 0:12:42 | 9.2 | 9.0 | 9.51 | 21.96 | 103% | 4.07 | 4.83 | 43% | 01:20 | 78% |
| WD28 | 22/12/02 03:26 | 0:13:58 | 5.8 | 2.0 | 6.62 | 5.52 | 114% | 4.78 | 1.61 | 72% | 00:28 | 71% |
| WD29 | 23/12/02 10:04 | 1:02:08 | 15.4 | 9.0 | 17.53 | 13.13 | 114% | 9.98 | 3.68 | 57% | | 72% |
| WD30 | 26/12/02 16:12 | 1:01:01 | 10.0 | 3.0 | 7.34 | 3.13 | 73% | 4.13 | 0.99 | 56% | 01:04 | 68% |
| WD31 | 29/12/02 06:54 | 0:04:32 | 2.4 | 6.0 | 2.61 | 9.49 | 109% | 1.02 | 1.79 | 39% | 00:13 | 81% |
| WD32 | 01/01/03 09:48 | 0:17:30 | 13.8 | 6.0 | 12.43 | 3.91 | 90% | 6.83 | 1.79 | 55% | 00:04 | 54% |
| WD33 | 08/01/03 11:28 | 0:01:58 | 3.2 | 12.0 | 2.61 | 4.81 | 82% | 0.79 | 1.38 | 30% | | 71% |
| WD34 | 18/01/03 19:26 | 0:14:22 | 6.0 | 2.0 | 4.40 | 3.79 | 73% | 1.91 | 1.61 | 43% | | 57% |
| WD35 | 20/01/03 11:14 | 0:10:38 | 8.6 | 3.0 | 10.06 | 10.20 | 117% | 4.26 | 5.12 | 42% | 00:01 | 50% |
| WD36 | 21/01/03 23:28 | 0:16:08 | 16.6 | 6.0 | 20.58 | 18.02 | 124% | 9.27 | 5.37 | 45% | | 70% |
| WD37 | 26/01/03 03:24 | 0:03:12 | 4.2 | 3.0 | 3.81 | 7.51 | 91% | 1.52 | 2.75 | 40% | 00:22 | 63% |
| WD38 | 28/01/03 17:48 | 0:15:18 | 18.8 | 18.0 | 19.88 | 34.04 | 106% | 10.32 | 9.12 | 52% | | 73% |
| **Min** | | **0:01:22** | **2.0** | **1.2** | **1.27** | **1.95** | **27%** | **0.70** | **0.81** | **30%** | **00:01** | **-16%** |
| **Max** | | **1:13:04** | **18.8** | **42.0** | **20.58** | **34.04** | **124%** | **15.84** | **9.12** | **123%** | **01:23** | **81%** |
| **Mean** | | **0:08:43** | **7.7** | **10.0** | **6.61** | **7.62** | **81%** | **4.27** | **3.67** | **66%** | **00:27** | **47%** |

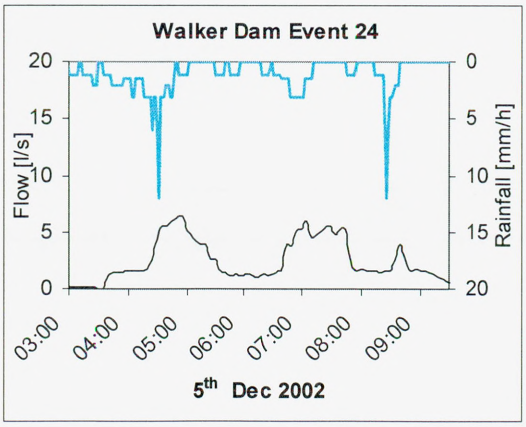
## C.4.2 Typical Hydrographs, Walker Dam

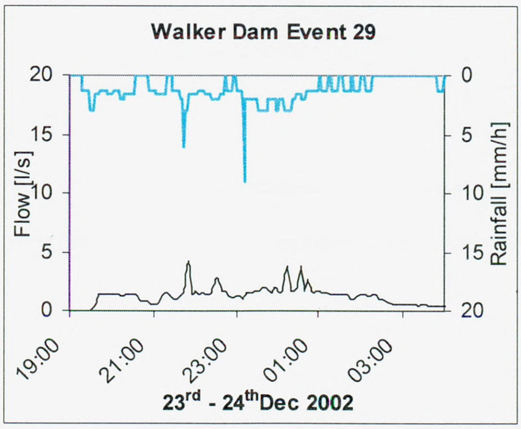


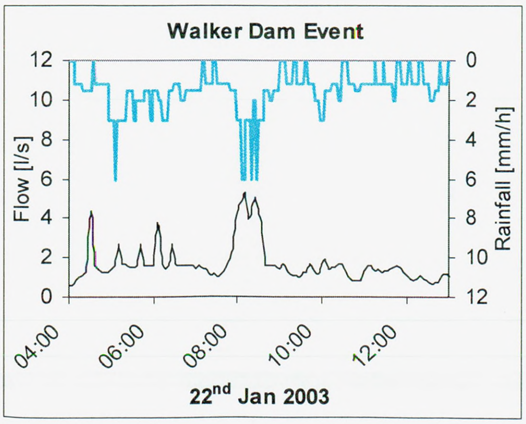



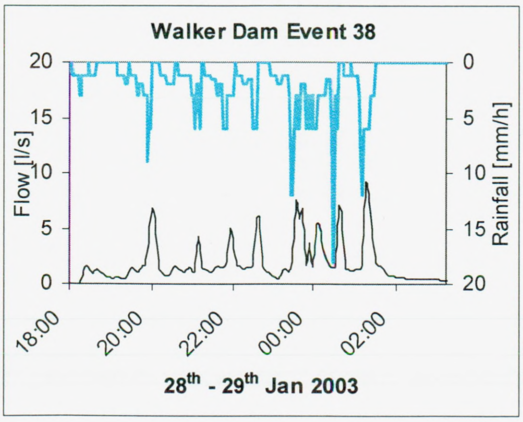



## C.4.3 Comparison of selected Parameters, Walker Dam

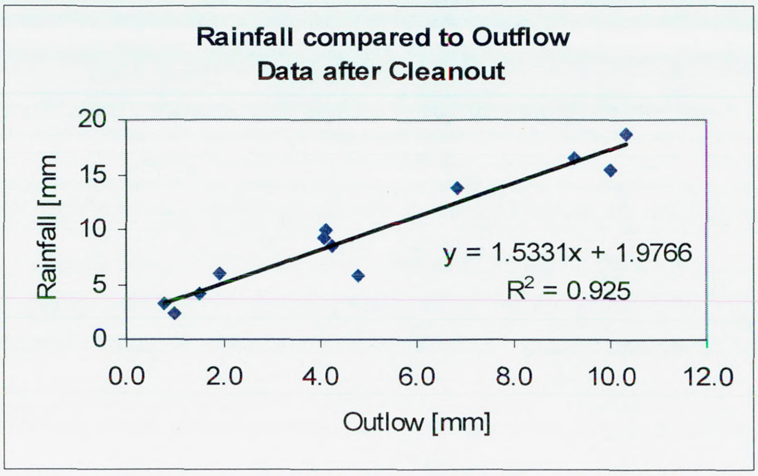



**Figure C-10: Rainfall compared with Flow**

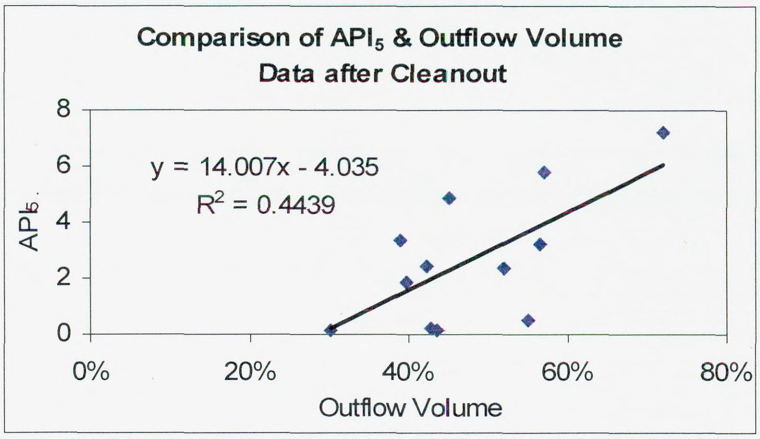



**Figure C-11: $API_5$ compared with Percentage Flow Volume**

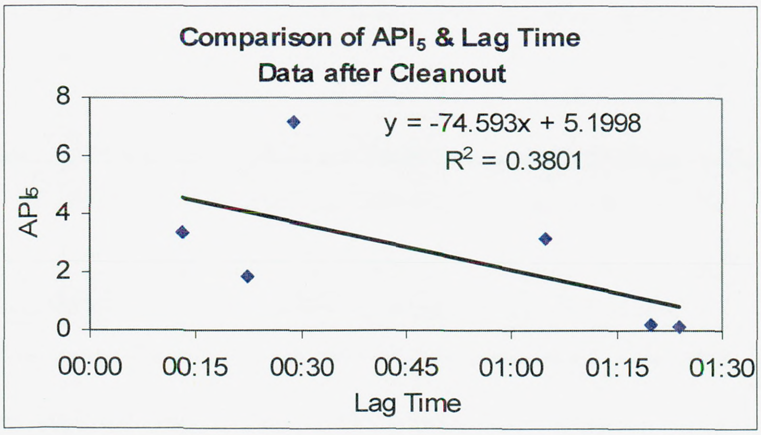


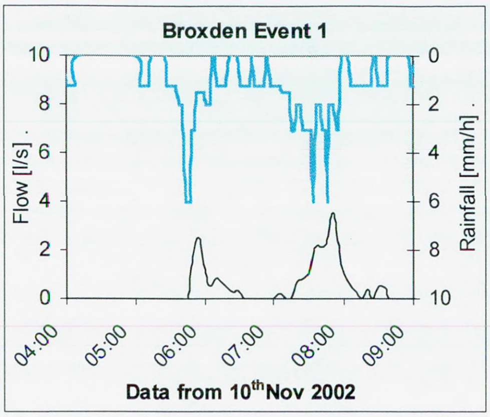

**Figure C-12: $API_5$ compared with Lag Time**

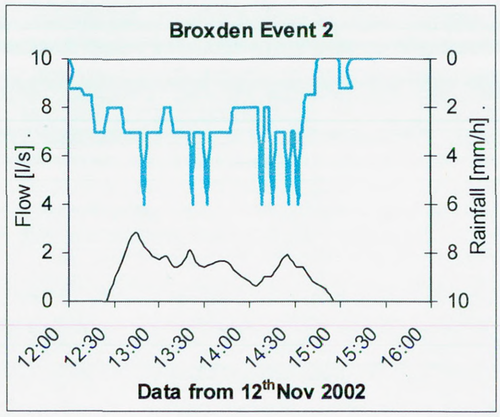
# Appendix - C.5 Monitoring Results, Broxden

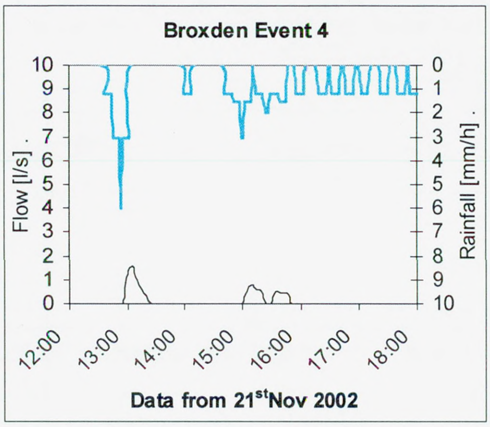
## C.5.1 List of Events

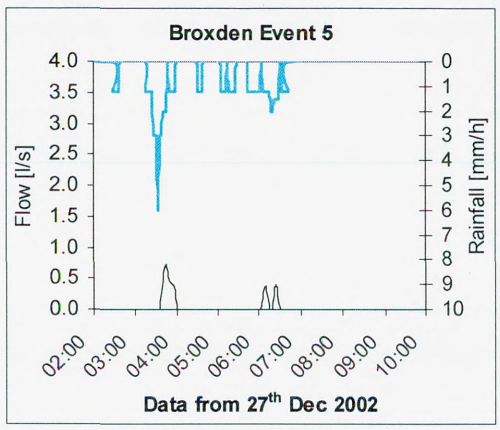
| Event | Rainfall | | | | Trench Inflow | | | Trench Outflow | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Rain Start | Duration | Total | Peak | Total Flow | Peak Flow | Flow Vol. | Total Flow | Peak Flow | Flow Vol. | Lag Time** | Peak flow reduction |
| [-] | [dd:hh:mm] | [d:hh:mm] | [mm] | [mm/h] | [mm] | [l/s] | [%] | [mm] | [l/s] | [%] | [hh:mm] | [%] |
| B1 | 10/11/02 04:00 | 0:05:06 | 5.6 | 6.0 | 5.29 | 8.72 | 95% | 1.31 | 3.56 | 25% | 00:14 | 59% |
| B2 | 12/11/02 12:00 | 0:03:08 | 7.6 | 6.0 | 7.14 | 7.62 | 94% | 1.91 | 2.87 | 27% | 00:02 | 62% |
| B3 | 20/11/02 10:56 | 0:05:50 | 5.2 | 2.0 | 6.02 | 6.11 | 116% | 1.16 | 1.06 | 19% | 00:27 | 83% |
| B4 | 21/11/02 09:00 | 0:15:00 | 11.2 | 6.0 | 8.55 | 5.87 | 76% | 2.09 | 1.60 | 24% | 00:28 | 73% |
| B5 | 27/11/02 02:27 | 0:04:16 | 3.4 | 6.0 | 2.27 | 4.55 | 67% | 0.52 | 0.73 | 23% | 00:46 | 84% |
| B6 | 27/11/02 15:10 | 0:11:50 | 4.0 | 3.0 | 2.95 | 4.47 | 74% | 0.70 | 0.68 | 24% | 00:33 | 85% |
| B7 | 28/11/02 18:41 | 0:08:49 | 2.6 | 2.0 | 1.30 | 3.39 | 50% | 0.45 | 0.68 | 35% | 00:25 | 80% |
| B8 | 30/11/02 09:37 | 0:09:18 | 3.6 | 2.0 | 2.23 | 3.30 | 62% | 0.68 | 0.50 | 30% | 00:22 | 85% |
| B9 | 01/12/02 06:43 | 0:06:22 | 10.0 | 6.0 | 6.18 | 11.00 | 62% | 1.29 | 2.24 | 21% | 00:25 | 80% |
| B10 | 04/12/02 00:09 | 0:11:01 | 4.6 | 3.0 | 3.45 | 5.40 | 75% | 1.01 | 1.17 | 29% | 00:56 | 78% |
| B11 | 06/12/02 01:20 | 0:04:00 | 2.4 | 3.0 | 2.49 | 5.20 | 104% | 0.59 | 1.27 | 24% | 00:46 | 76% |
| B12 | 07/12/02 15:28 | 0:19:18 | 4.8 | 3.0 | 3.68 | 5.03 | 77% | 1.52 | 1.26 | 41% | 00:47 | 75% |
| B13 | 14/01/03 12:05 | 1:08:40 | 8.6 | 30.0 | 5.99 | 12.84 | 70% | 1.19 | 2.62 | 20% | 00:21 | 80% |
| B14 | 24/01/03 09:25 | 0:21:50 | 3.4 | 1.5 | 2.87 | 3.07 | 84% | 0.72 | 0.48 | 25% | 00:21 | 84% |
| B15 | 27/01/03 12:20 | 0:18:09 | 4.6 | 6.0 | 3.53 | 8.28 | 77% | 0.93 | 1.16 | 26% | 00:11 | 86% |
| B16 | 01/02/03 17:30 | 0:22:31 | 15.4 | 12.0 | 7.53 | 9.31 | 49% | 2.29 | 3.44 | 30% | 00:25 | 63% |
| **Min** | | **0:03:08** | **2.4** | **1.5** | **1.30** | **3.07** | **49%** | **0.45** | **0.48** | **19%** | **00:02** | **59%** |
| **Max** | | **1:08:40** | **15.4** | **30.0** | **8.55** | **12.84** | **116%** | **2.29** | **3.56** | **41%** | **00:56** | **86%** |
| **Mean** | | **0:12:26** | **6.1** | **6.1** | **4.47** | **6.51** | **77%** | **1.15** | **1.58** | **27%** | **00:28** | **77%** |

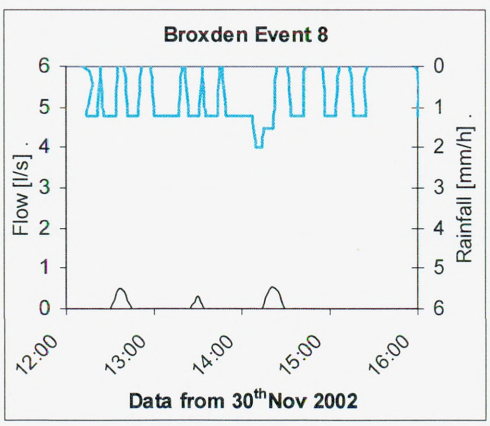
## C.5.2 Typical Hydrographs, Broxden

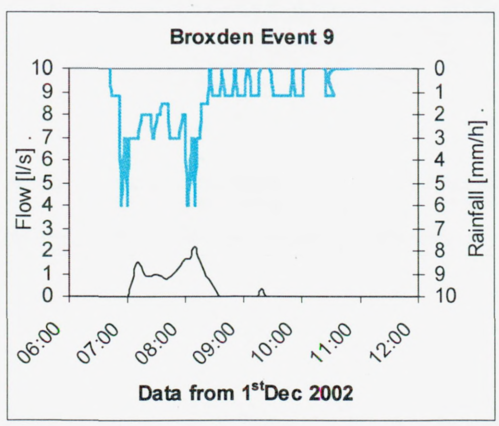


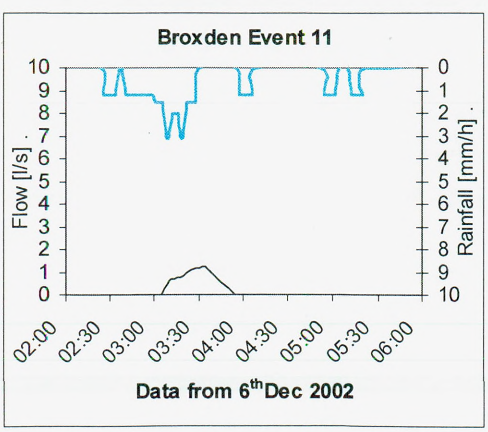



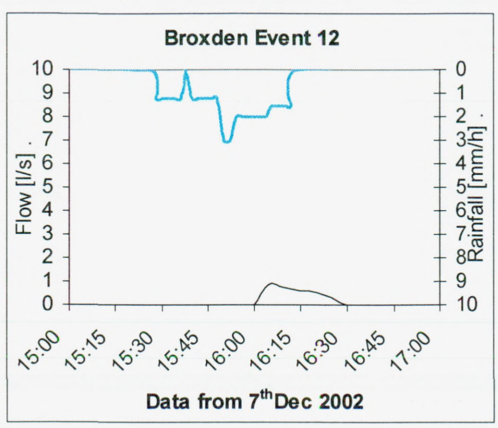



### C.5.3 Comparison of selected Parameters, Broxden

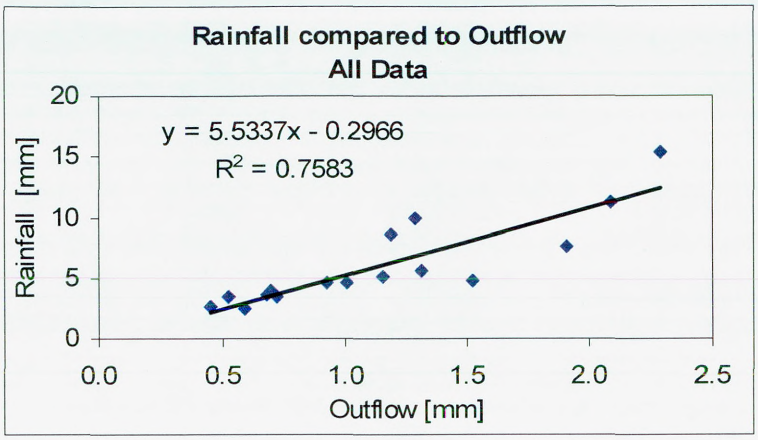



**Figure C-13: Rainfall compared with Outflow**

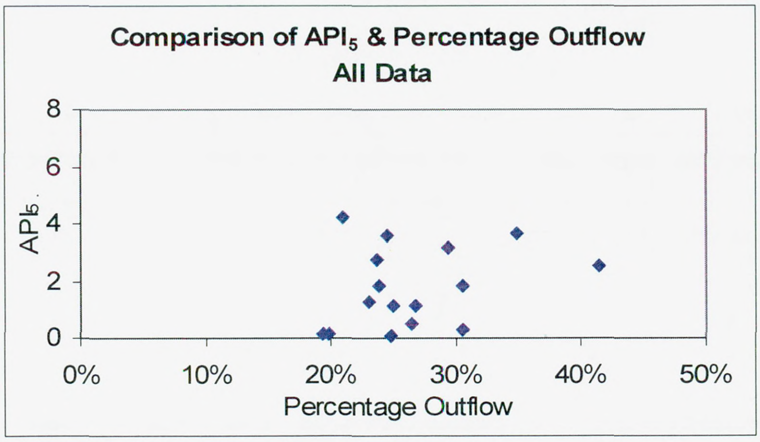



**Figure C-14: $API_5$ compared with Percentage Flow Volume**

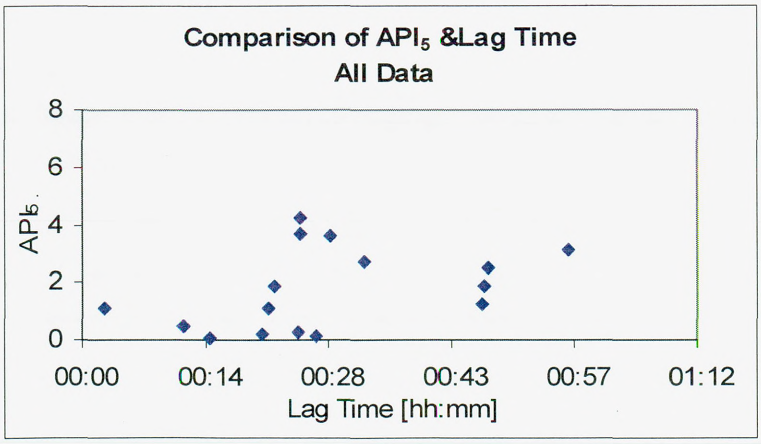



**Figure C-15: $API_5$ compared with Lag Time**

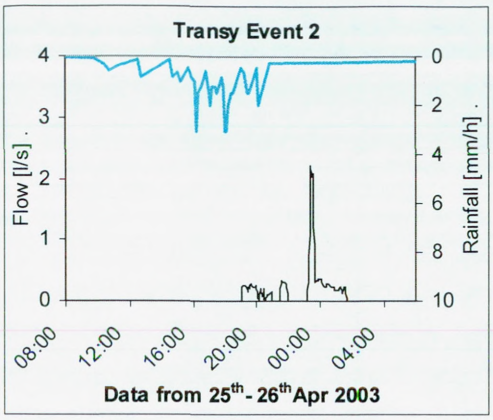
# Appendix - C.6 Monitoring Results, Transy

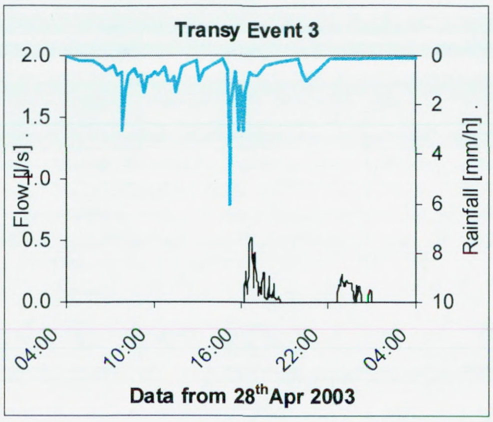
## C.6.1 List of Events

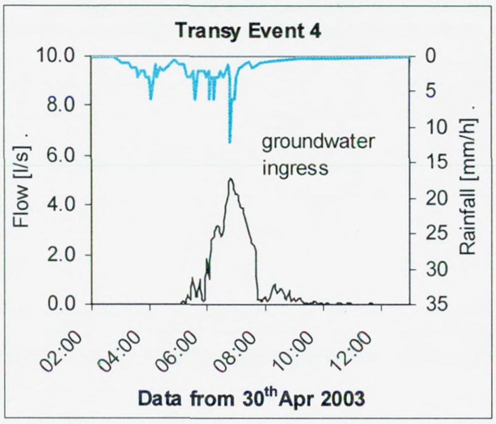
| | Rainfall | | | | Trench Inflow | | | Trench Outflow | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Event | Rain Start | Duration | Total | Peak | Total Flow | Peak Flow | Flow Volume | Total Flow | Peak Flow | Flow Volume | Lag Time* | Peak flow reduction |
| [-] | [dd:hh:mm] | [d:hh:mm] | [mm] | [mm/h] | [mm] | [l/s] | [%] | [mm] | [l/s] | [%] | [hh:mm] | [%] |
| TD1 | 31/03/2003 23:13 | 0:05:22 | 3.8 | 3.0 | 0.82 | 3.27 | 22% | 0.00 | 0.00 | 0% | N/A | 100% |
| TD2 | 25/04/2003 06:00 | 0:14:50 | 7.8 | 6.0 | 0.81 | 0.92 | 10% | 1.22 | 2.23 | 152% | N/A | -142% |
| TD3 | 28/04/2003 04:56 | 0:17:14 | 9.2 | 12.0 | 0.92 | 1.31 | 10% | 0.64 | 0.53 | 70% | 06:26 | 60% |
| TD4 | 30/04/2003 02:48 | 0:06:26 | 10.2 | 24.0 | 2.59 | 10.57 | 25% | 3.77 | 5.09 | 146% | N/A | 52% |
| TD5 | 30/04/2003 13:02 | 0:08:52 | 21.4 | 6.0 | 3.84 | 10.17 | 18% | 7.81 | 6.23 | 203% | N/A | 39% |
| TD6 | 01/05/2003 06:40 | 0:04:34 | 3.0 | 0.8 | 0.73 | 2.46 | 24% | 1.00 | 2.65 | 138% | N/A | -8% |
| TD7 | 03/05/2003 17:06 | 0:02:00 | 2.6 | 3.0 | 0.42 | 1.66 | 16% | 0.00 | 0.00 | 0% | N/A | 100% |
| TD8 | 04/05/2003 19:50 | 0:07:20 | 4.6 | 3.0 | 0.66 | 2.03 | 14% | 0.27 | 0.70 | 42% | 01:26 | 66% |
| TD9 | 10/05/2003 22:08 | 0:03:18 | 2.8 | 6.0 | 1.21 | 11.43 | 43% | 0.00 | 0.00 | 0% | N/A | 100% |
| TD10 | 11/05/2003 18:10 | 0:04:54 | 4.6 | 6.0 | 0.27 | 1.01 | 6% | 0.08 | 0.12 | 28% | 02:04 | 88% |
| TD11 | 14/05/2003 15:54 | 0:03:04 | 6.6 | 15.0 | 0.17 | 0.53 | 3% | 0.00 | 0.00 | 0% | N/A | 100% |
| TD12 | 16/05/2003 18:58 | 0:02:16 | 2.2 | 4.0 | 0.38 | 1.39 | 17% | 0.00 | 0.00 | 0% | N/A | 100% |
| TD13 | 16/05/2003 21:42 | 0:03:52 | 3.4 | 6.0 | 0.42 | 1.52 | 12% | 1.11 | 0.00 | 264% | N/A | 100% |
| TD14 | 18/05/2003 18:52 | 0:01:06 | 5.6 | 60.0 | 1.07 | 14.87 | 19% | 1.66 | 0.53 | 155% | N/A | 96% |
| TD15 | 19/05/2003 15:32 | 0:01:26 | 2.6 | 9.0 | 0.35 | 3.78 | 13% | 0.27 | 2.65 | 77% | 00:28 | 30% |
| TD16 | 08/06/2003 05:14 | 0:03:44 | 5.2 | 4.0 | 0.66 | 1.01 | 13% | 0.08 | 0.00 | 12% | 04:17 | 100% |
| TD17 | 09/06/2003 21:18 | 0:14:40 | 5.6 | 6.0 | 0.64 | 0.57 | 11% | 0.00 | 0.00 | 0% | N/A | 100% |
| TD18 | 12/06/2003 02:26 | 0:08:32 | 3.6 | 24.0 | 0.21 | 0.45 | 6% | 0.00 | 0.58 | 0% | N/A | -29% |
| TD19 | 27/06/2003 17:52 | 0:10:06 | 8.4 | 6.0 | 1.64 | 3.26 | 19% | 0.60 | 0.00 | 37% | 03:20 | 100% |
| **Min** | | **0:01:06** | **2.2** | **0.8** | **0.2** | **0.5** | **3%** | **0.0** | **0.0** | **0%** | **00:28** | **-142%** |
| **Max** | | **0:17:14** | **21.4** | **60.0** | **3.8** | **14.9** | **43%** | **7.8** | **6.2** | **264%** | **06:26** | **100%** |
| **Mean** | | **0:06:30** | **6.0** | **10.7** | **0.9** | **3.8** | **16%** | **1.0** | **1.1** | **70%** | **03:00** | **61%** |

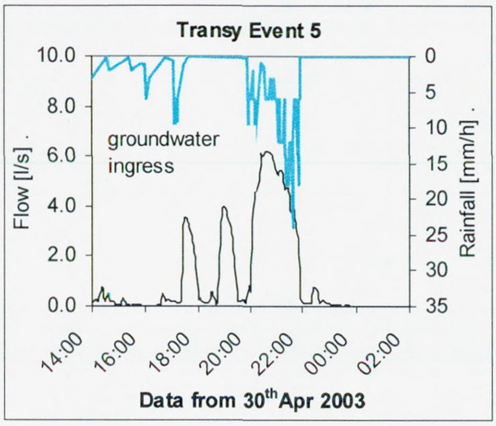
*N/A refer to events that produced zero outflow or were excluded from analysis due to groundwater ingress

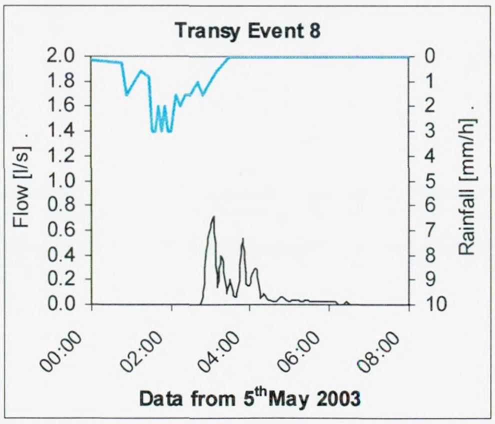
## C.6.2 Typical Hydrographs, Transy

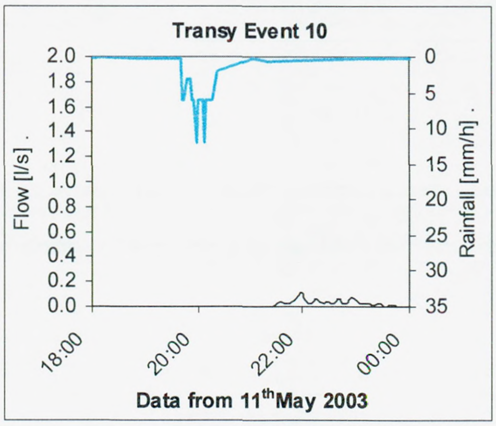


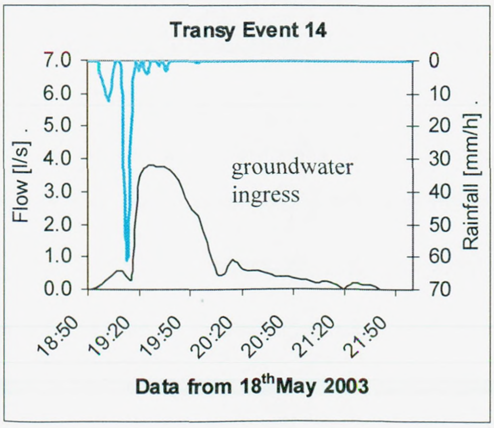



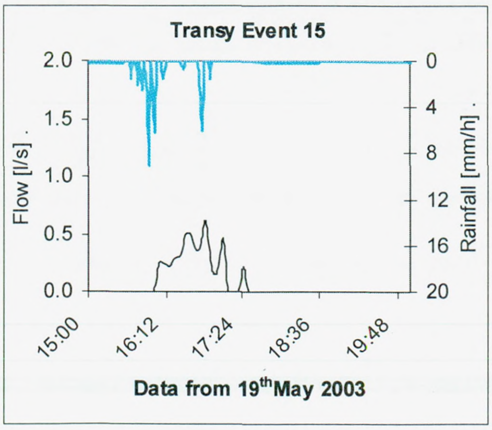



## C.6.3 Comparison of selected Parameters, Transy

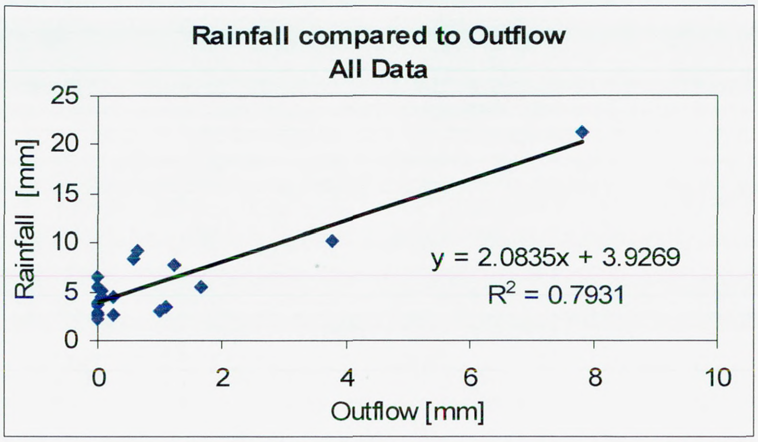



**Figure C-16: Rainfall compared with to Outflow**

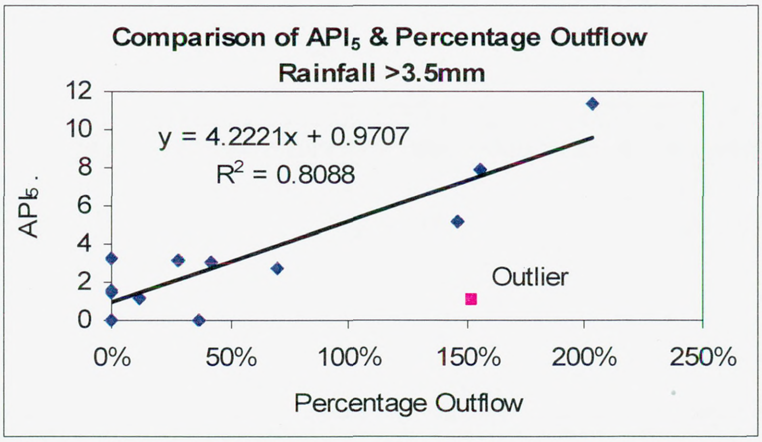



**Figure C-17: $API_5$ compared with Percentage Flow Volume**

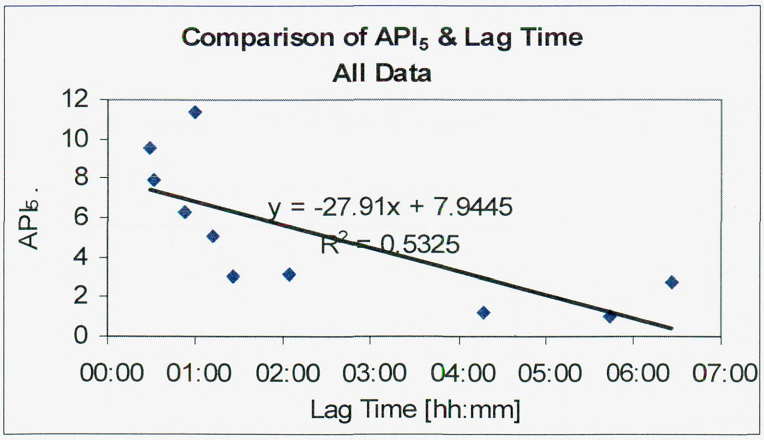


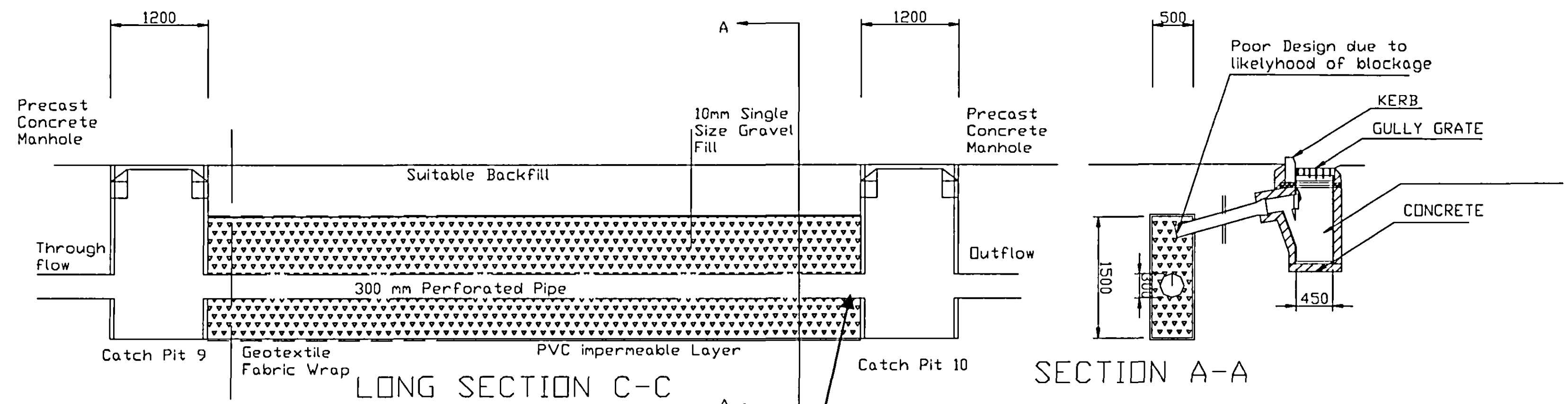

**Figure C-18: $API_5$ compared with Lag Time**

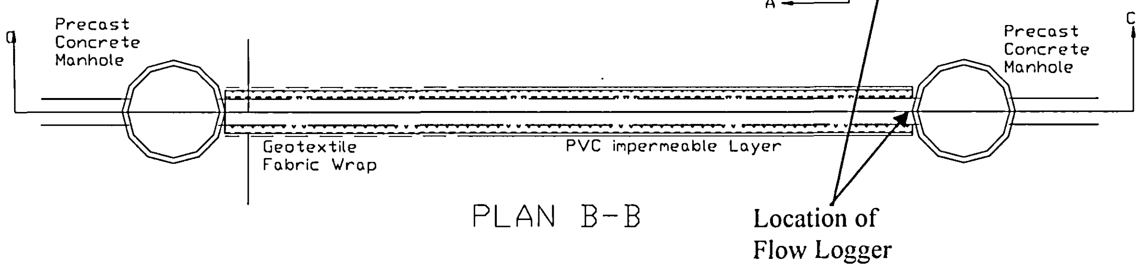
## Appendix - D Construction Drawings of monitored systems and their Performance Rating

| | | |
|---|---|---|
| **D.1** | **Filter Drain at Lang Stracht, Aberdeen** | **Rated Failure** |
| **D.2** | **Filter Drain at Spine Road, Dunfermline** | **Rated Fair** |
| **D.3** | **Filter Drain along A90 Glencarse** | **Rated Good** |
| **D.4** | **Infiltration Trench at Walker Dam, Aberdeen** | **Rated Fair** |
| **D.5** | **Infiltration Trench at Broxden, Perth** | **Rated Excellent** |
| **D.6** | **Infiltration Trench at Transy, Dunfermline** | **Rated Good** |

**Note: For definition of rating system see Section 4.4**

# Appendix - D.1 Filter Drain at Lang Stracht, Aberdeen Rated Failure




NOTES:

STONE FILL:
CLEAN STONE 100MM SINGLE SIZE BROKEN STONE IN TRENCH 2M WIDE 1.2M DEEP PLACED ABOVE GRAVEL FILL

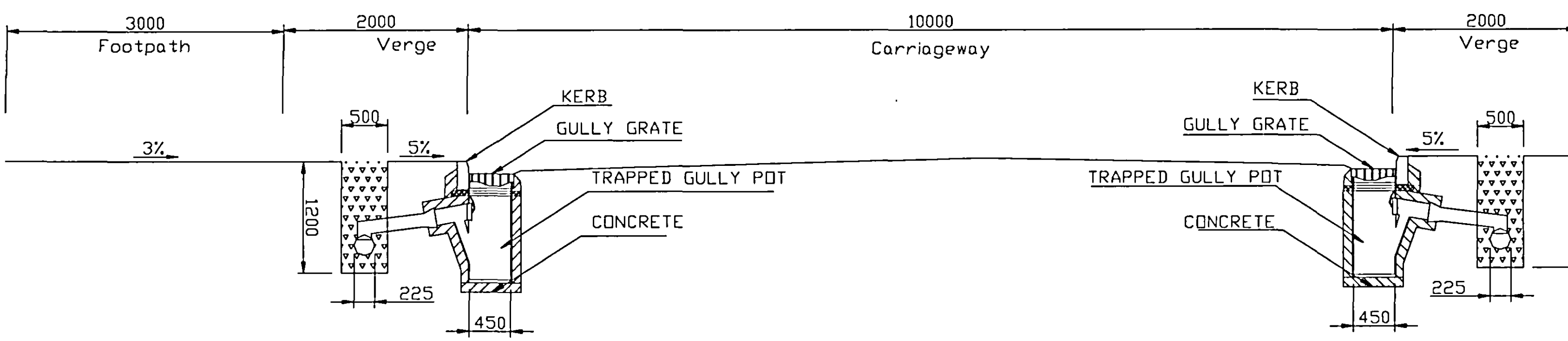
GRAVEL FILL:
CLEAN STONE 25MM SINGLE SIZE FILTER STONE GRAVEL FILL PLACED AS 200MM THICK BED AND SURROUND TO PERFORATED PIPE

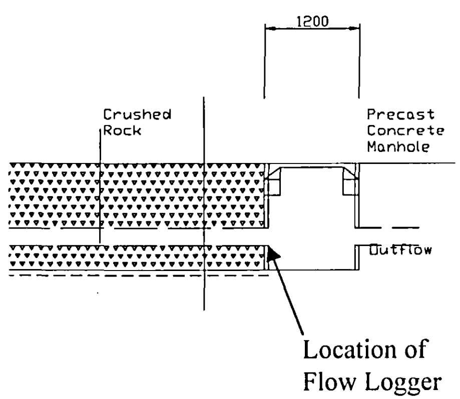
GEOTEXTILE FABRIC:
STONE AND GRAVEL TO BE WRAPPED IN TERRAM 700MM OR EQUIVALENT

| | |
|---|---|
| Project: Roadside Filter Drain Lang Stracht, Aberdeen | |
| Date 12/20/2002 | Scale N/A |
| Author: W.Schluter | |
| Drawing: **Construction Details** | |

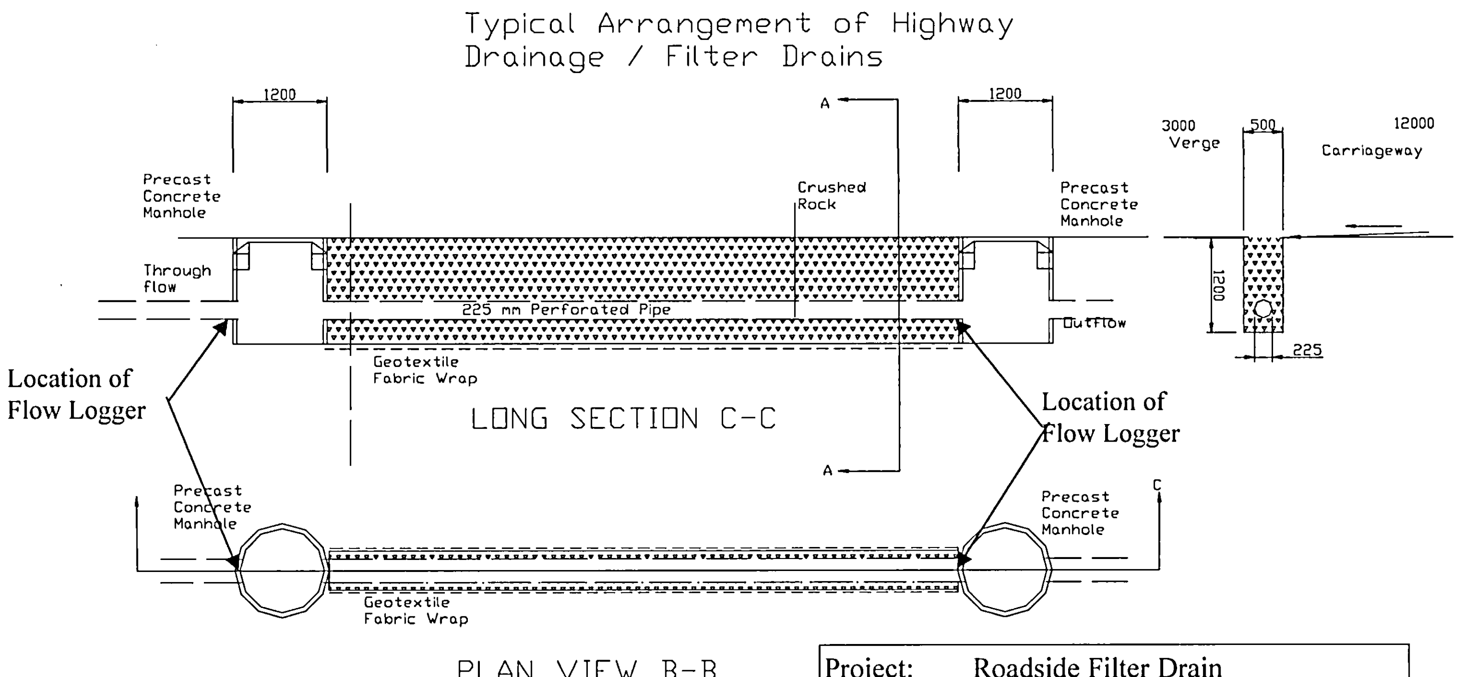
Appendix - D.2 Filter Drain at Spine Road, Dunfermline Rated Fair

# Typical Arrangement of Highway Drainage / Filter Drains



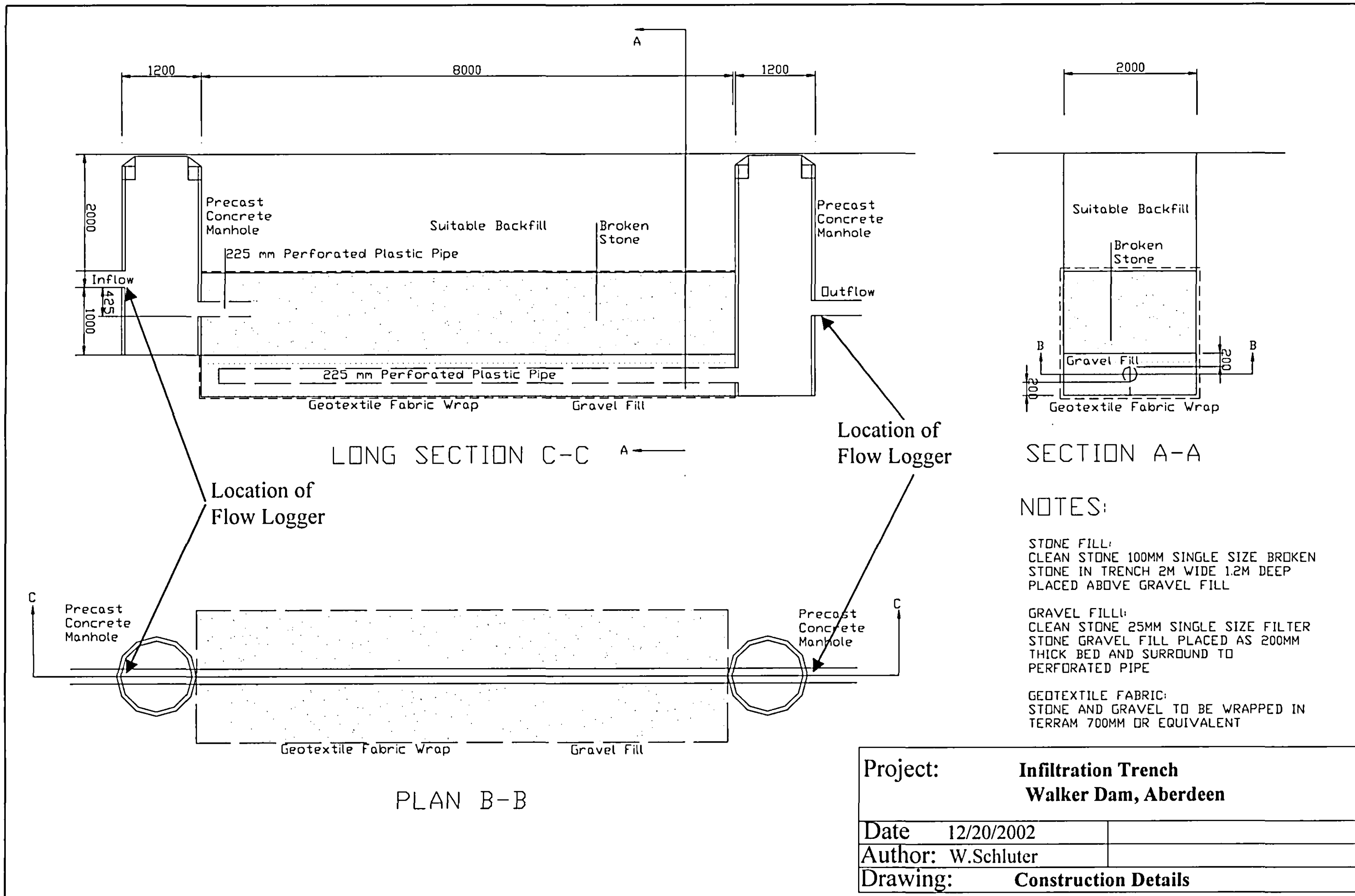




| | | |
|---|---|---|
| Project: | Roadside Filter Drain<br>Spine Road South/Central, DEX | |
| Date | 16/04/2003 | Scale N/A |
| Author: | W.Schluter | |
| Drawing: | **Construction Details** | |

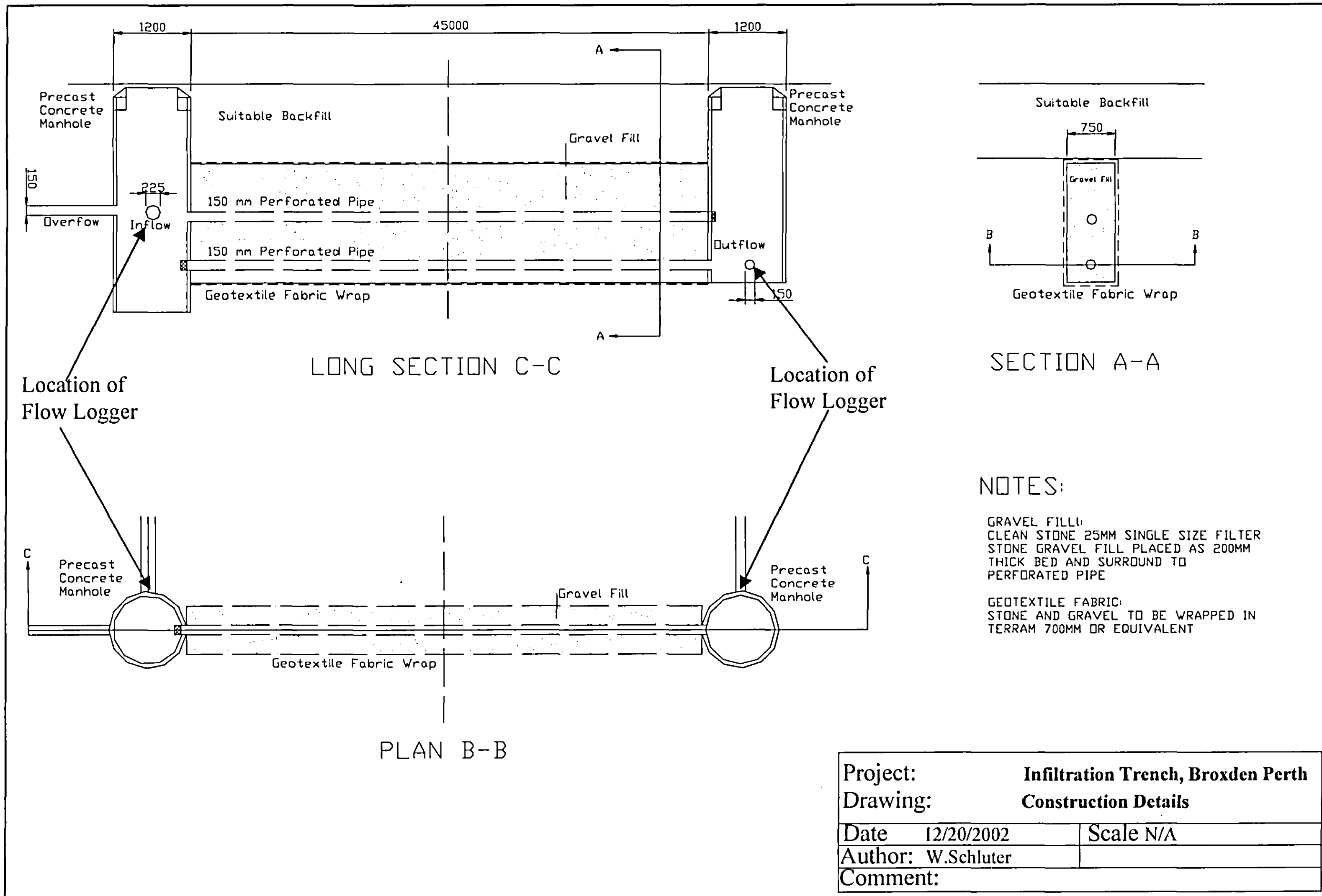



| | | | |
|---|---|---|---|
| Project: | Roadside Filter Drain<br>Section along A90, Glencarse | | |
| Date | 29/07/2003 | Scale N/A | |
| Author: | W.Schluter | | |
| Drawing: | **Construction Details** | | |

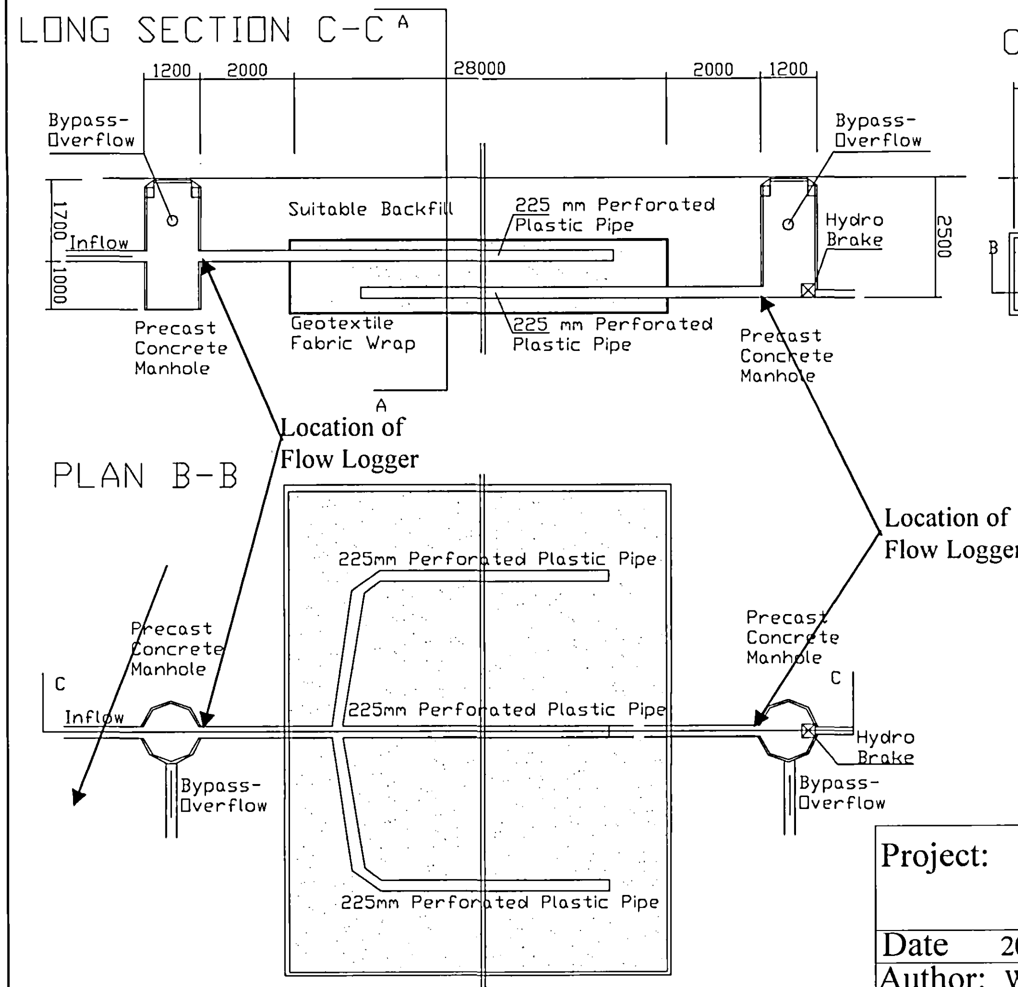
# Appendix - D.4 Infiltration Trench at Walker Dam, Aberdeen Rated Fair

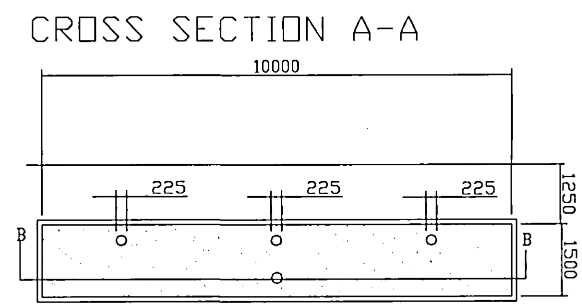


# Appendix - D.5 Infiltration Trench at Broxden, Perth Rated Excellent



## Appendix - D.6 Infiltration Trench at Transy, Dunfermline Rated Good




NOTES:

1. ALL PIPES TO BE UPVC ULTRA RIB BY MARLEY OR SIMILAR.
2. OVERFLOW PIPE TO BE INSTALLED BETWEEN MANHOLES S8 AND S26 AS SHOWN
3. STORAGE AREA TO BE 10M X 28M X 1.5M.
4. FILTER MATERIAL TO BE 100MM - 150MM NOMINAL SINGLE SIZED CRUSHED ROCK.
5. CRUSHED ROCK ALL CONTAINED WITHIN A 1000 GAUGE TERRAM FABRIC OR EQUIVALENT.

| | |
|---|---|
| Project: | **Infiltration Trench Transy Estate, Dunfermline** |
| Date | 20/04/2003 |
| Author: | W.Schluter |
| Drawing: | **Construction Details** |

# Appendix - E Information from all Investigated Sites

## E.1 Summary of Information from all Sites

## E.2 Detailed evaluation of the System's Performance

| Number | Locatin | Site | Layout Plans | Detailed Construction Drawings | Design Calculation | Catchment Area | Catchment Type* | Houses connected | Type of SUD System** | No of SUDS Pipe | Diameter of SUD pipe | Width | Length | Height | Largest Diameter | Treatment Volume | Treatment Volume/ Area | Depth below ground | Receiving Water | Overflow | Type of Inlet*** | Date of construction |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| [-] | [-] | [-] | [-] | [-] | [-] | [-] | [-] | [-] | [m²] | [-] | [-] | [-] | [-] | [mm] | [m] | [m] | [m] | [mm] | [m³] | [m³/m²] | [m] | [-] |
| 1 | Aberdeen | Lang Stracht | Yes | Yes | Yes | 9520 | MR | 0 | FD | 1 | 300 | 0.5 | 500 | 1.5 | 300 | 137.2 | 14 | 1.85 | Sewer System | Yes | TGP | Aug-98 |
| 2 | Aberdeen | Woodend | Yes | No | Yes | 13000 | H,LR | 42 | EP IT | 2 | 400 | X | X | X | 450 | X | X | X | Burn of Rubislaw | No | TGP | Aug-98 |
| 3 | Aberdeen | Arnhall Business Park | Yes | No | No | X | LR | 0 | FD | 0 | X | X | X | X | X | X | X | X | Burn | X | TGP | 2001 |
| 4 | Aberdeen | Elrick Road | No | No | No | X | LR | 0 | FD | 0 | X | X | X | X | X | X | X | X | Burn | X | TGP | 1999 |
| 5 | Aberdeen | Arnhall Business Park | Yes | No | No | X | CP | X | FD | X | X | X | X | X | X | X | X | X | X | X | OK | 2000 |
| 6 | Aberdeen | Great Northern Rd | No | No | No | X | F | X | FD | X | X | X | X | X | X | X | X | X | X | X | LF | X |
| 7 | Aberdeen | Walkerdam | Yes | Yes | No | 3424.5 | H,LR | 15 | EP IT | 1 | 225 | 2 | 8 | 1.5 | 225 | 7.4 | 2 | 3.75 | Walker Dam (Lake) | No | TGP | Aug-00 |
| 8 | Blairhall | Housing Estate | Yes | Yes | Yes | 1285 | H,LR | 6 | EP IT | 1 | 225 | 8 | 8 | 1.25 | 225 | 24.2 | 19 | 1.75 | Burn | Yes | TGP | 1999 |
| 9 | Dundee | Swallow Roundabout | No | No | No | X | MR | 0 | FD | 1 | 225 | X | X | X | 225 | X | X | 1.5 | Burn | No | TGP | 1994 |
| 10 | Dunfermline | Spine Rd, DEX | Yes | Yes | No | 3057 | LR | 0 | FD | 1 | 150 | 0.5 | 808 | 1.2 | 150 | 155.4 | 51 | 1.2 | LyneBurn | No | TGP | 1999 |
| 11 | Dunfermline | Transy Estate | Yes | Yes | Yes | 6568 | H,LR | 15 | EP IT | 3 | 225 | 10 | 28 | 1.5 | 225 | 126.8 | 19 | 2.7 | Sewer | Yes | TGP | Jan-99 |
| 12 | Dunfermline | Queens Gate | Yes | No | Partial | 200000 | H,LR | 158 | EP IT | 0 | 0 | 2 | 52 | 2.2 | X | 68.6 | 0.3 | 3 | Sewer System | Yes | TGP | X |
| 13 | Dunfermline | Woodmill Road | Yes | No | No | 460 | LR | 0 | FD | 1 | 150 | 0.5 | 5 | X | 150 | X | X | 1.2 | Sewer System | No | TGP | Oct-98 |
| 14 | Dunfermline | Tesco Car park, DEX | No | No | No | X | F | 0 | FD | 1 | 150 | X | X | X | X | X | X | 1.75 | Burn | No | LF | X |
| 15 | Dunfermline | Spine Rd, DEX | Yes | Yes | No | X | LR | 0 | FD | 1 | 150 | 0.5 | 255 | 1.2 | 150 | 49.1 | X | 1.2 | LyneBurn | No | OK | 1999 |
| 16 | Dunfermline | Pitreavie | No | No | No | X | LR | X | FD | 0 | X | X | X | X | X | X | X | X | X | X | TGP | X |
| 17 | Edinburgh | Forthroad bridge North | No | No | No | X | MR | 0 | FD | 1 | 225 | X | X | X | 225 | X | X | 1.5 | Burn | Yes | LF | X |
| 18 | Edinburgh | Forthroad bridge South | No | No | No | X | MR | 0 | FD | 1 | 225 | X | X | X | 225 | X | X | 1.5 | Burn | Yes | LF | X |
| 19 | Falkirk | RetailPark at Queens St | Yes | Yes | Yes | 21000 | CP | 0 | FD | 1 | 150 | X | X | X | 375 | X | X | X | Sewer System | Yes | TGP | 2000 |
| 20 | Falkirk | Wallacelea Rumford | Yes | Yes | partial | 3400 | LR | 0 | EP IT | 1 | 300 | 3.5 | 19.5 | 1.675 | 300 | 35.3 | 10 | 3 | Burn | Yes | TGP | Feb-00 |
| 21 | Falkirk | Maddiston California Rd | Yes | Yes | Partial | 5651 | H,LR | X | FD | 1 | 250 | 3 | 20 | 1.5 | 300 | 72.0 | 13 | 2.1 | Burn | No | TGP | May-00 |
| 22 | Falkirk | Wagon Road, Polmont | Yes | Yes | Yes | 7120 | H,LR | 180 | EP IT | 1 | 225 | X | X | 1.5 | 150 | X | X | 2.4 | Burn | Yes | TGP | Jul-03 |
| 23 | Falkirk | BusinessPark, Larbert | Yes | Yes | No | 2280 | CP | 0 | FD | 1 | 150 | 1 | 47 | 1 | 150 | 14.7 | 6 | 1 | Sewer System | No | LF | Mar-98 |
| 24 | Falkirk | Braes Highschool, Polmont | Yes | Yes | Yes | 5000 | H,LR | 0 | FD | 1 | 225 | 3.5 | 34 | 1.25 | 225 | 45.6 | 9 | 1.4 | Burn | Yes | TGP | X |
| 25 | Falkirk | Queens Drive Larbert | Yes | no | no | X | CP, LR | 0 | FD | 2 | 150 | X | X | X | 150 | X | X | 1.65 | Sewer System | No | TGP | 1997 |
| 26 | Falkirk | Glenbo, Dennyloanhead | Yes | no | no | X | H,LR | 37 | EP IT | X | X | X | X | X | X | X | X | X | Sewer System | No | X | Jul-94 |
| 27 | Falkirk | Taymouth Rd, Heatherlea | Yes | Yes | No | 935 | LR | 0 | FD | 2 | 150 | 0.6 | 4 | 1 | 150 | 0.8 | 1 | X | Sewer System | Yes | TGP | 1998 |
| 28 | Findochty | Netherton Farm | Yes | Yes | Yes | 675 | H,LR | X | EP IT | 1 | 150 | 5 | 10.6 | 1.2 | X | 19.2 | 28 | 2.55 | Sewer System | Yes | TGP | Oct-00 |
| 29 | Hatton | Lairds Park, Fintray | Yes | Yes | Yes | 7400 | H,LR | 23 | S, FD | 3 | 3x225 | 10 | 30 | 2 | 300 | 182.5 | 25 | 2.75 | Soil Infiltration | Yes | TGP | Jul-99 |
| 30 | Invergorwie | Invergorwie Mill | Yes | No | No | X | H,LR | X | EP IT, S | 0 | X | X | X | X | X | X | X | X | Burn | X | TGP | Aug-00 |
| 31 | Kirkhill | Albyn | Yes | Yes | Yes | 9644 | H,LR | 25 | S, FD | 1 | 450 | 0.9 | 35 | 1.5 | 400 | 59.6 | 6 | 3.1 | Burn | No | TGP | Feb-01 |
| 32 | Perth | Broxden | Yes | Yes | Yes | 3300 | H,LR | 14 | EP IT | 2 | 150 | 0.75 | 45 | 1.5 | 150 | 15.7 | 5 | 2 | Craigie Burn | Yes | TGP | Jun-99 |
| 33 | Perth | Glencarse | No | No | No | 7061.5 | MR | 0 | FD | 1 | 225 | 0.5 | 717 | 1.2 | 225 | 149.0 | 21 | 1.2 | Burn | No | LF | X |
| 34 | Perth | St Madoes Old | Yes | Yes | No | 2129 | LR | 0 | EP IT | 1 | 150 | 1 | 48 | X | X | X | X | 1 | Cairnie Pow | No | TGP | 1997 |
| 35 | Perth | St Madoes New | Yes | No | No | X | H,LR | X | FD | X | X | X | X | X | X | X | X | X | Burn | No | TGP | 2001 |
| 36 | Perth | Faries Rd | Yes | No | No | 3872 | H,LR | Varies | FD | 0 | X | 1.5 | 72 | X | X | X | X | X | X | X | TGP | Nov-00 |
| 37 | Tayport | Sandy Hill | Yes | No | No | 392 | LR | 0 | FD | 1 | 150 | 2 | 55 | 0.75 | 150 | 25.4 | 65 | X | Burn | Yes | TGP | Feb-99 |
| 38 | Tayport | Sandy Hill, S1 | Yes | No | No | 2205 | H,LR | 20 | S | 0 | 0 | 10 | 22 | X | 225 | X | X | X | Burn | Yes | TGP | 2000 |
| 39 | Tayport | Sandy Hill, S8 | Yes | No | No | 2016 | H,LR | 21 | S | 1 | 225 | 9 | 22 | X | 225 | X | X | X | Burn | Yes | TGP | 2000 |
| 40 | Tillicoultry | Tillyflats Grangemouth | Yes | No | No | X | LR | 0 | FD | X | X | X | X | X | X | X | X | X | Burn | X | X | 1999 |
| 41 | Tullicoultry | Fir Park | Yes | Yes | Yes | 3610 | LR | 0 | EP IT | 2 | 225 | 1.5 | 33 | 1.75 | 225 | 26.9 | 7 | 3.5 | Burn | Yes | TGP | May-99 |
| 42 | Westhill | Leddach Farm New | Yes | No | Yes | 2200 | H,LR | 9 | EP IT | 1 | 225 | 0.75 | 85 | X | 150 | X | X | X | Burn | Yes | TGP | Sep-01 |
| 43 | Westhill | Leddach Farm Old | Yes | No | No | X | H,LR | X | EP IT | 1 | 450 | X | X | X | X | X | X | 2 | Burn | X | TGP | X |

| Type of Catchment | | | | | | Type of SUD System | | | | Type of Inlet | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| * | MR Major Road | LR Local Road | CP Car Park | H House Area | F Field | ** | EP IT End-of-pipe Infiltration Trench | RS FD Road side Filter Drain | S Soakaway | *** | TGP Typical Trapped Gully Pot | OK Offlet Kerb | LF Lateral Flow |

# Appendix - E.2 Detailed evaluation of the System Ratings

See Section 4.4 for definition of rating system

| Number | Site, Location | Overall Rating | Category 1 | Category 2 | Category 3 | Category 4 | Category 5 | Water Quality Performance | Hydraulic Performance | Comments |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Lang Stracht, Aberdeen | 1.0 | 2 | 1* | 1 | 1 | 3 | CCTV showed no accumulation of sediments in perforated pipe. System failed after 2.5 years of operation, producing highly turbid discharge | Failed, due to frequent overspilling of gully pots | No sedimentation within perforated pipe. 20-50% of gullie: blocked. High turbid outflow after gully cleaning |
| 2 | Woodend, Aberdeen | 1.0 | 3 | 2 | 1* | 2 | 5 | Settling out of sediments rather than filtering due to direct connection of in- and outlet | Negligible inifltration due to soil characteristics and high ground water level, acts as storage chamber, no throttle | Designed with low confidence in functionality. Cleaning requi Twin perforated pipes were approximately 2/3 blocked: Visibl both catch pits |
| 3 | Arnhall Business Park, Aberdeen | X | | | | | | X | X | X |
| 4 | Elrick Road, Aberdeen | X | | | | | | X | X | No information was received |
| 5 | Arnhall Business Park, Aberdeen | X | | | | | | X | X | No information was received |
| 6 | Great Northern Rd, Aberdeen | X | | | | | | X | X | No information was received |
| 7 | Walkerdam, Aberdeen | 3.6 | 3 | 4 | 5 | 2 | 4 | Settling out of sediments in the inlet chamber, filtering in trench due to disconnected system. Fine particles were found in outlet of the system, which could get flushed into the receiving water | Frequent surcharge of inlet manhole prior to cleanout. This was due to the partially blocked inlet. No surcharge recorded after cleanout. And also improve attenuation in terms of infiltrated flow volume. | X |
| 8 | Housing Estate, Blairhall | 4.0 | 5 | 5 | 4 | 3 | 3 | Good water quality performance expected , as system has small catchment area, producing little amount of pollution. T-piece in addition to perforated pipe should ensure filtering through of inflow | Hydraulic performance is ensured using a perforated pipe that is followed through the trench. A dip plate is installed in the inlet chamber to reduce the velocity and hold back floating debris. A T-piece is installed at the upper end of the perforated pipe to increas the inflow into the filter medium. | Outlet pipe into burn may be to shallow and allow for water in from burn. 225 diameter t-piece should perform well for wat input into trench. Perforated pipe followed through whicch ens hydraulic performance, whereras disconnection would have provided better water quality. |
| 9 | Swallow Roundabout, Dundee | 1.0 | 1* | 3 | 1 | 1 | 5 | Poor Water Quality performance due to heavy pollution loading | Depends on ground condition (infiltration) Measurements of similar desgin showed good attenuation | Heavy sedimentation at several locations |
| 10 | Spine Rd, DEX, Dunfermline | 3.2 | 3 | 3 | 3 | 3 | 4 | Poor Water Quality performance. Turbidity measurement showed increased readings downstream | Good flow attenuation. The top section performs better then bottom section, which may be due to the change in slop from 3.5% to 1.5%. | X |
| 11 | Transy Estate, Dunfermline | 4.4 | 5 | 5 | 4 | 5 | 3 | Good water quality performance expected , good design and large treatment volume | Good flow attenuation, occasionally influenced by ground water incress | Good design but cleaning facilites of perforated pipes could h been included, i.e. rodding eyes |
| 12 | Queens Gate, Dunfermline | 1.0 | 1* | 2 | 2 | 1 | 2 | No treatment as all flow bypasses the trench. Trench inlet is completey blocked. | None | System connected prior termination of construction. Cleanir required. Designed to treat first 10% of Rain. |
| 13 | Woodmill Road, Dunfermline | 3.0 | 3 | 3 | 3 | 3 | 3 | Outlet chamber use good pipe arrangement for particle settlement also to hold back floating debris. | Gully pots were full of debris, which reduces flow capacity | X |
| 14 | Tesco Car park, DEX, Dunfermline | 1.0 | 1* | 3 | 1 | 1 | 3 | Poor Water Qualiy performance as heavy sedimentation input from field runoff | 0 | Heavy sedimentation in inspection chambers and also visible or of the field drain |
| 15 | Spine Rd, DEX, Dunfermline | 2.4 | 2 | 4 | 4 | 1 | 1 | Water quality expected to be better than section with gully pots as flow is filtered through the trench medium. | Several locations were identified were inlets (offlet kerbs) were blocked, resulting in flooding | Offlet kerbs were chosen and seem to block easily for this application. There is also a problem with the stability of these i |
| 16 | Pitreavie, Dunfermline | X | | | | | | X | X | X |
| 17 | Forth road bridge North, Edinburgh | 2.6 | 3 | 3 | 1 | 3 | 3 | Poor Water Quality performance due to heavy pollution loading | Depends on ground condition (infiltration) Measurements of similar desgin showed good attenuation | Heavy sedimentation at several locations |
| 18 | Forth road bridge South, Edinburgh | 2.6 | 3 | 3 | 1 | 3 | 3 | Poor Water Quality performance due to heavy pollution loading | Depends on ground condition (infiltration) Measurements of similar desgin showed good attenuation | Heavy sedimentation at several locations |
| 19 | Retail Park at Queens St, Falkirk | X | | | | | | X | X | Construction differ from drawings as no overflow pipe fron trenches are visible. |
| 20 | Wallacelea Rumford, Falkirk | 2.4 | 2 | 2 | 3 | 2 | 3 | Settling out of sediments rather than filtering due to direct connection of in- and outlet. Both short end perforated pipe surcharged. Overflow is probably taken most inflow | Acts as storage chamber, no throttle | Lower pipes probably blocked or poor infiltration and permane surcharged |
| 21 | Maddiston California Rd, Falkirk | X | | | | | | X | X | Could not find inspection chambers either covered or nonexis |
| 22 | Wagon Road, Polmont, Falkirk | X | | | | | | X | X | Housing construction ongoing at time of site visit 25/07/03. I access to system. |
| 23 | BusinessPark, Larbert, Falkirk | 3.4 | 4 | 4 | 3 | 3 | 3 | Good water quality expected, as inflow is filtered through material. Sump within filter drain could have been larger for higher sediment storage | Inflow into trench has to be kept clean. Local blockage visible, which may cause problems | Site appeared untidy and aesthecially unpleasant. Porous pave prefered. However system appears to be working satisfactor |
| 24 | Braes Highschool, Polmont, Falkirk | X | | | | | | X | X | Could not find inspection chambers, probably covered |

* Critera resulted in system failure

| Number | Site, Location | Overall Rating | Category 1 | Category 2 | Category 3 | Category 4 | Category 5 | Water Quality Performance | Hydraulic Performance | Comments |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | Queens Drive Larbert, Falkirk | X | | | | | | X | X | No inspection chambers designed/ constructed. |
| 26 | Glenbo, Dennyloanhead, Falkirk | X | | | | | | X | X | SUDS device not on plans provided |
| 27 | Taymouth Rd, Heatherlea, Falkirk | 3.0 | 2 | 3 | 3 | 3 | 4 | Good water quality performance expected , as system has small catchment area, producing little amount of pollution | X | Short Roadside trenches receiving road runoff via typical TGPs. Sediment traps contain a significant amount of sediments. Filters seem to be working OK. Gully pots need cleaning. USED 1 as typical |
| 28 | Netherton Farm, Findochty | X | | | | | | X | X | Housing construction ongoing at time of site visit. Water standing in system, indicating low permeable soil. |
| 29 | Lairds Park, Fintray, Hatton | 3.6 | 3 | 4 | 3 | 5 | 3 | All inflow seems to infiltrate. Overflow not used | No sign of flooding. | System connected prior termination of construction. Cleaning required.Catchpit was filled up. Perforated pipes were 1/3 blocked. Ground conditions suitable.Heavy sedimentation from construction runoff will reduce the longetivity significantly. |
| 30 | Invergowrie Mill, Invergowrie | X | | | | | | X | X | Construction ongoing at the time of inspection |
| 31 | Albyn, Kirkhill | 2.4 | 2 | 2 | 2 | 3 | 3 | X | X | Housing construction ongoing at time of site visit. Water standing in system, indicating low permeable soil. |
| 32 | Broxden, Perth | 4.0 | 5 | 5 | 5 | 4 | 5 | Settling out of sediments in the inlet chamber, filtering in trench due to disconnected system. Catchpits were found clean at inspection. Good Water Quality performance | System seems to work as expected. No sign of flooding. Good hydraulic performance. Overflow active once during monitoring, as a result of a heavy storm | System was relatively new and appeared to be performing well |
| 33 | Glencarse, Perth | 4.0 | 5 | 4 | 3 | 4 | 4 | Poor Water Quality performance due to heavy pollution loading and no sediment trapping prior to inflow | No sign of flooding or sedimentation. Measurement suggests good flow attenuation | X |
| 34 | St Madoes Old, Perth | 4.0 | 4 | 1 | 3 | 3 | 3 | No sediment sump provided and perforated pipe is probably function as bypass. | No sign of flooding or sedimentation | Onsite observations did not match construction drawings |
| 35 | St Madoes New, Perth | X | | | | | | X | X | Construction differ from drawings provided. Early connection of drainage resulted in complete blockage at this site., |
| 36 | Faries Rd, Perth | X | | | | | | X | X | Site under construction |
| 37 | Sandy Hill, Tayport | 2.4 | 2 | 2 | 3 | 3 | 2 | Good water quality performance expected if soil condition suitable. Large treatment volume! Site inspection showed unexpected findings off heavy pollution discharging from one trench, whereas the other seems to be operating satisfactory. | Overflow ensures hydraulic performance. Seems to be operating freqently for one but not for the second system. | X |
| 38 | Sandy Hill, S1, Tayport | 3.0 | 2 | 3 | 4 | 3 | 3 | A better flow distribution would be advisable. Good water quality performance expected if soil condition suitable. | Overflow ensures hydraulic performance. | X |
| 39 | Sandy Hill, S8, Tayport | 2.8 | 2 | 3 | 4 | 2 | 3 | One leg of perforated pipes seems too little for the infiltration area. A better flow distribution would be advisable. | No sign of blockage | X |
| 40 | Tillyflats Grangemouth, Tillicoultry | X | | | | | | X | X | Site acces not granted due to security reasons |
| 41 | Fir Park, Tullicoultry | 1.0 | 1* | 1* | 1 | 1 | 3 | Settling out of sediments rather than filtering due to direct connection of in- and outlet. Extrem drop (1.5) of chamber inlet and trench inlet. Sump only 0.3m Bad design, poor sediment settling | Perforated pipe is blocked with sediments | Extrem drop (1.5) of chamber inlet and trench inlet. Sump only 0.3m Bad design, poor sediment settling |
| 42 | Leddach Farm New , Westhill | X | | | | | | Settling out of sediments rather than filtering due to direct connection of in- and outlet | Throttle in place to limit outlfow and use extra storage | System was relatively new and appeared to be performing well |
| 43 | Leddach Farm Old, Westhill | 2.0 | 3 | 2 | 1 | 1 | 3 | Settling out of sediments rather than filtering due to direct connection of in- and outlet. Fine particles accumulated at the outlet of the system | Acts as storage chamber, no throttle | Designed with low confidence in functionality. Cleaning required. |

* Critera resulted in system failure

# Appendix - F Code for FED Model in Hydrol Basic

```
;-----------------------------------------------------------------------
; VARIABLE DECLARATIONS (only specific parameters are shown, see Figure 6-6 for more ;detail
;-----------------------------------------------------------------------
const float Length=9.9
const float w=2
const float P=0.35
const float Kh= 0.01
const float Ks=1.5e-008
const float Kv= 1.3e-006
const int ArrayElements=4
const float HL=0.9
;------------------------------ ------------------------------------------------------------
;Initial Calculation
;------------------------------ ------------------------------------------------------------
        dx=Length/(ArrayElements)
;------------------------------ ------------------------------------------------------------
;Inflow is Given. Convert QIN to m3/s
;------------------------------ ------------------------------------------------------------
        Q[1]=Inflow/1000
;------------------------------ ------------------------------------------------------------
;Calculate Volume (V)and Water level (H) and Darcy Flow (Q) in Cells i
;------------------------------ ------------------------------------------------------------
        i=1
        Do While (i<ArrayElements)
        V[i]=V[i]+(Q[i]-Q[i+1])*STEP*3600-Vx[i+1]
        H[i] = V[i]/(w*dx*P)
        Q[i+1]=Kh*(H[i]+H[i+1])*0.5*w*(H[i]-H[i+1])/dx
                If (H[i]<=0) Then
                        H[i]=0
                        V[i]=0
                        Vx[i]=0
                        Else
                        Vx[i]=(w*dx*Kv) *STEP*3600+(Ks*H[i]*0.5*dx) *STEP*3600
                End If
        H[ArrayElements] = V[ArrayElements]/(w*dx*P)
        V[ArrayElements]=V[ArrayElements]+(Q[i]-Q[i+1])*STEP*3600-Vx[i+1]

;Lookup Outflow from H-Q Relationship stored in Library
                If (H[ArrayElements]<HL) Then
                        Qout=0
                        Else
                        X=H[ArrayElements]-HL
                        QLP=LOOKUP(SigLev_Flowdown(X))
;Limit Outflow to Maximum available outflow per TimeStep
                                If ((QLP*STEP*3600)>(X*dx*w*P)) Then
                                        Qout=(X*dx*w*P)/(STEP*3600)
                                        Else
                                        Qout=QLP
                                End If
                End If
        i = i + 1
        Loop
```

# Appendix - G Modelling Results using Erwin

**G.1 Hydraulic Modelling of Lang Stracht**

**G.2 Hydraulic Modelling of Spine Road**

**G.3 Hydraulic Modelling of Glencarse**

**G.4 Hydraulic Modelling of Broxden**

**G.5 Hydraulic Modelling of Walker Dam**

## *Appendix - G.1 Hydraulic Modelling of Lang Stracht*

The calibration procedure is outlined in Section 5.8.1 and this is also repeated here for ease of reference. The calibration of each model involved undertaking several runs with varying parameters to try and match the outflow hydrograph using an eyeball comparison. Default values as described in the Erwin User Manual (Ingenieurgesellschaft fuer Stadthydrology, 1997) were used as initial set-up and were then modified to achieve the best fit. To obtain a numerical comparison between the recorded and simulated volume, a methodology showing their average difference was also adopted (see Equation (G-1)). The equation incorporates the absolute value of the difference, ensuring positive and negative values do not cancel each other out (Nix, 1994), (Maksimovic and Radojkovic, 1986).

$$F = \frac{\sum |r_i|}{n} \qquad \textbf{(G-1)}$$

F [%] is the goodness of fit criterion, $r_i$ [%] is the difference between recorded and simulated volume per event and n is the number of events. The list of parameters that were available to calibrate the models is shown in Table G-1.

The system at Lang Stracht can be simulated satisfactory when selecting a limited period of up to 6 months with an F value of 26%. Unfortunately using the same parameters for the whole monitoring period does not produce satisfactory results, F increases to over 50%. This may be due to the characteristics of the model, which uses partial gully pot blockage as mechanism for flow attenuation and this may not be represented accurately long-term. Figure G-1 show a comparison of event volume show a comparison of typical hydrographs.

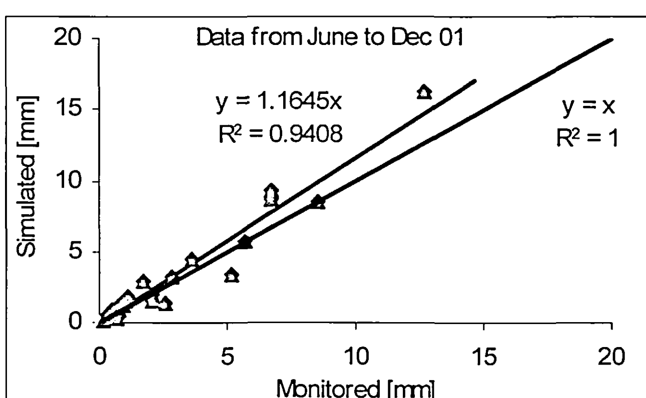



**Figure G-1: Comparison of Event Volume at Lang Stracht Aberdeen**

The gully pot inlets were represented in the model and used to limit the inflow into the system, which resulted in the frequent overspill. The head discharge relationships of the gully inlets were found to have the main influence on the system's flow characteristic. Infiltration rates at this site were extremely low and porosity was set to 30%. Table G-1 shows calibration parameters to simulate flow at Lang Stracht.

| Parameter | Unit | Value |
|---|---|---|
| Initial Loss | [mm] | 0.5 |
| Depression Storage | [mm] | 0.9 |
| Initital Runoff | [%] | 20% |
| Final Runoff | [%] | 80% |
| Porosity of Fill Material | [%] | 30% |
| Infiltration Rate Base | [m/s] | 0 |
| Infiltration Rate Sides | [m/s] | $2\ 10^{-10}$ |

**Table G-1: Main calibration parameters to model Lang Stracht**

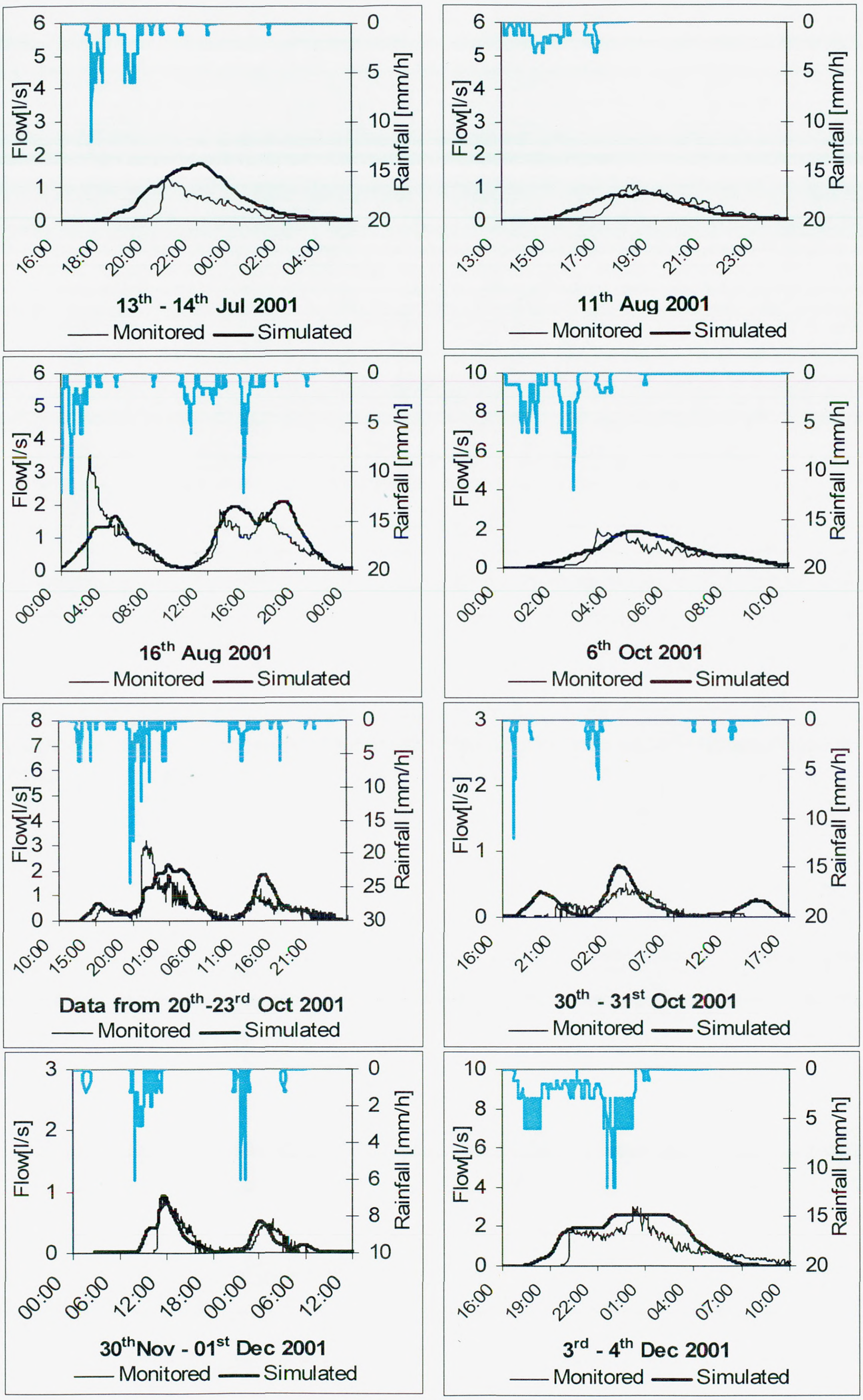



**Figure G-2: Hydrographs of simulated and monitored outflow at Lang Stracht**

## *Appendix - G.2 Hydraulic Modelling of Spine Road*

A good representation of the flow behaviour at Glencarse was achieved. The F-value was in the order of 24%. This improves to less than 19%, when discarding the worst event. Figure G-5 shows the results comparing monitored and simulated flow volume and Figure G-6 shows typical flow events.

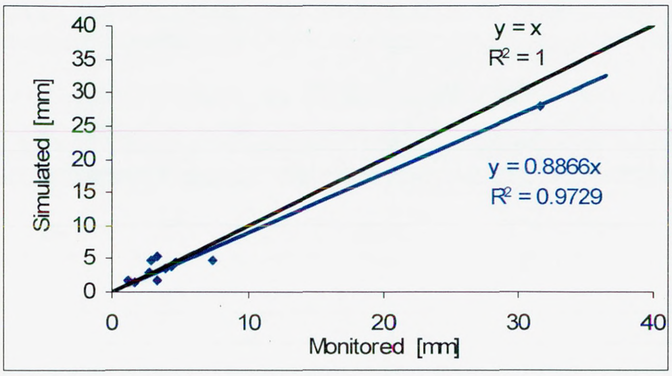



**Figure G-3: Comparison of Event Volume at Filter Drain Spine Road**

The system at Spine Road was characterised by a high percentage runoff and little initial losses. The system's volume reduction is mainly through exfiltration losses. Table G-2 shows the main calibration parameters.

| Parameter | Unit | Value |
|---|---|---|
| Initial Loss | [mm] | 0.4 |
| Depression Storage | [mm] | 0.4 |
| Initital Runoff | [%] | 30% |
| Final Runoff | [%] | 95% |
| Porosity of Fill Material | [%] | 30% |
| Infiltration Rate Base | [m/s] | $1\ 10^{-6}$ |
| Infiltration Rate Sides | [m/s] | $2\ 10^{-6}$ |

**Table G-2: Main calibration parameters to model Spine Road**

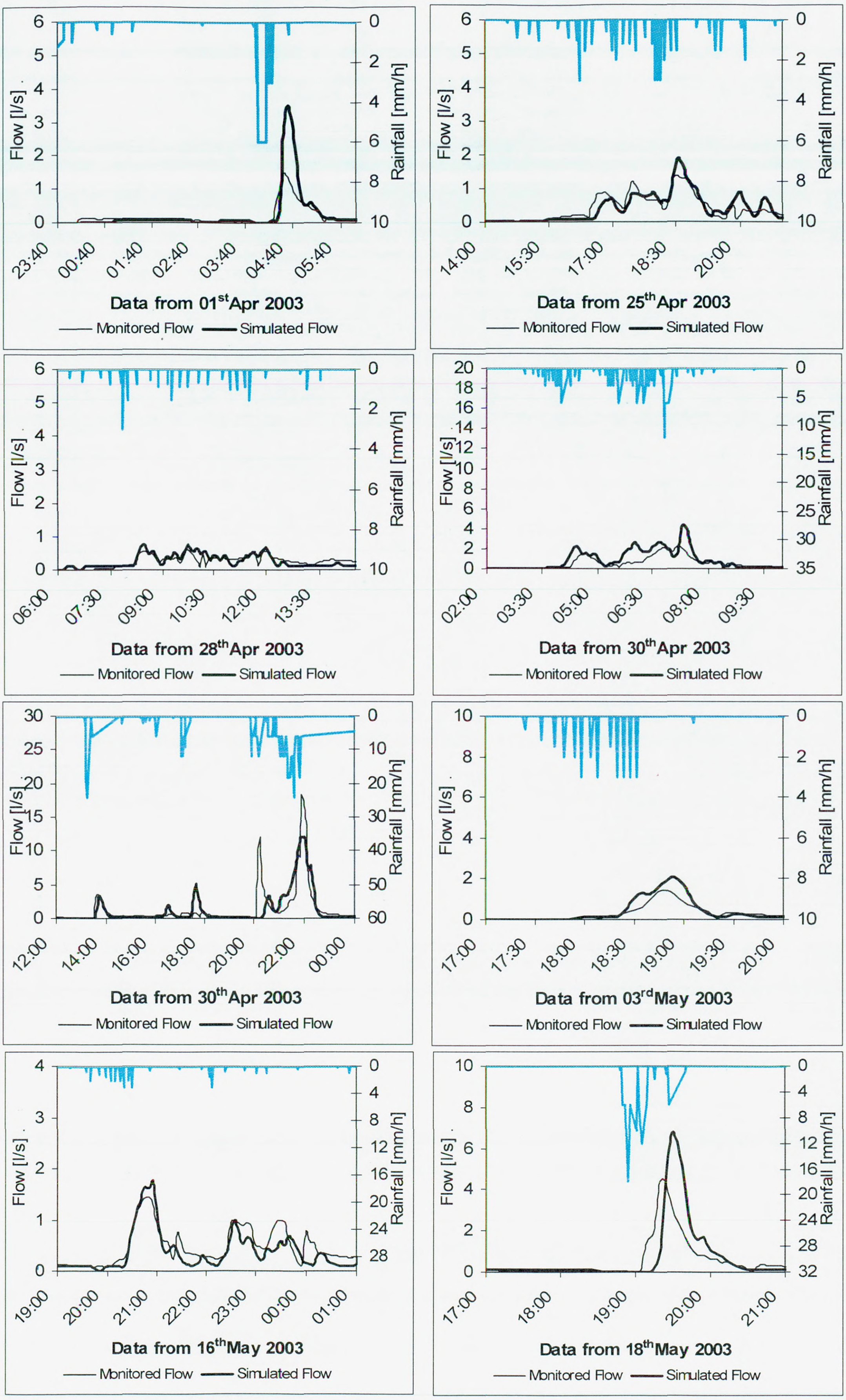



**Figure G-4: Hydrographs of simulated and monitored outflow at Spine Road**

## *Appendix - G.3 Hydraulic Modelling of Glencarse*

A good representation of the flow behaviour at Glencarse was achieved. The F-value was in the order of 20%. Figure G-5 shows the results comparing monitored and simulated flow volume and Figure G-6 shows typical flow events.

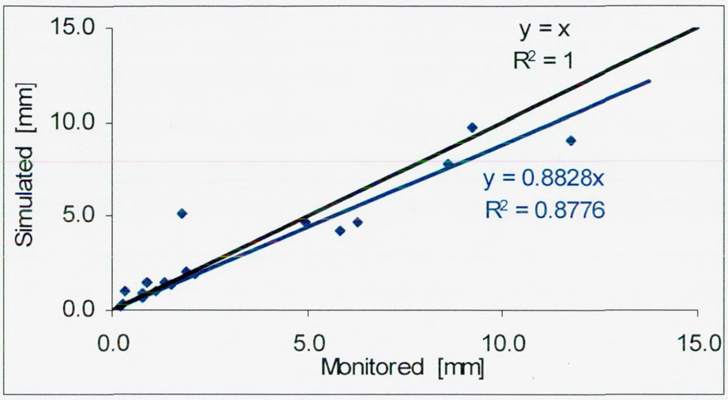



**Figure G-5: Comparison of Event Volume at Filter Drain Glencarse**

The system at Glencarse was characterised by a relatively quick response and little initial losses. The system's volume reduction is mainly through exfiltration losses, which were the most important calibration parameters.

| Parameter | Unit | Value |
|---|---|---|
| Initial Loss | [mm] | 0.4 |
| Depression Storage | [mm] | 0.4 |
| Initital Runoff | [%] | 25% |
| Final Runoff | [%] | 90% |
| Porosity of Fill Material | [%] | 30% |
| Infiltration Rate Base | [m/s] | $1\ 10^{-6}$ |
| Infiltration Rate Sides | [m/s] | $2\ 10^{-6}$ |

**Table G-3: Main calibration parameters to model Glencarse**

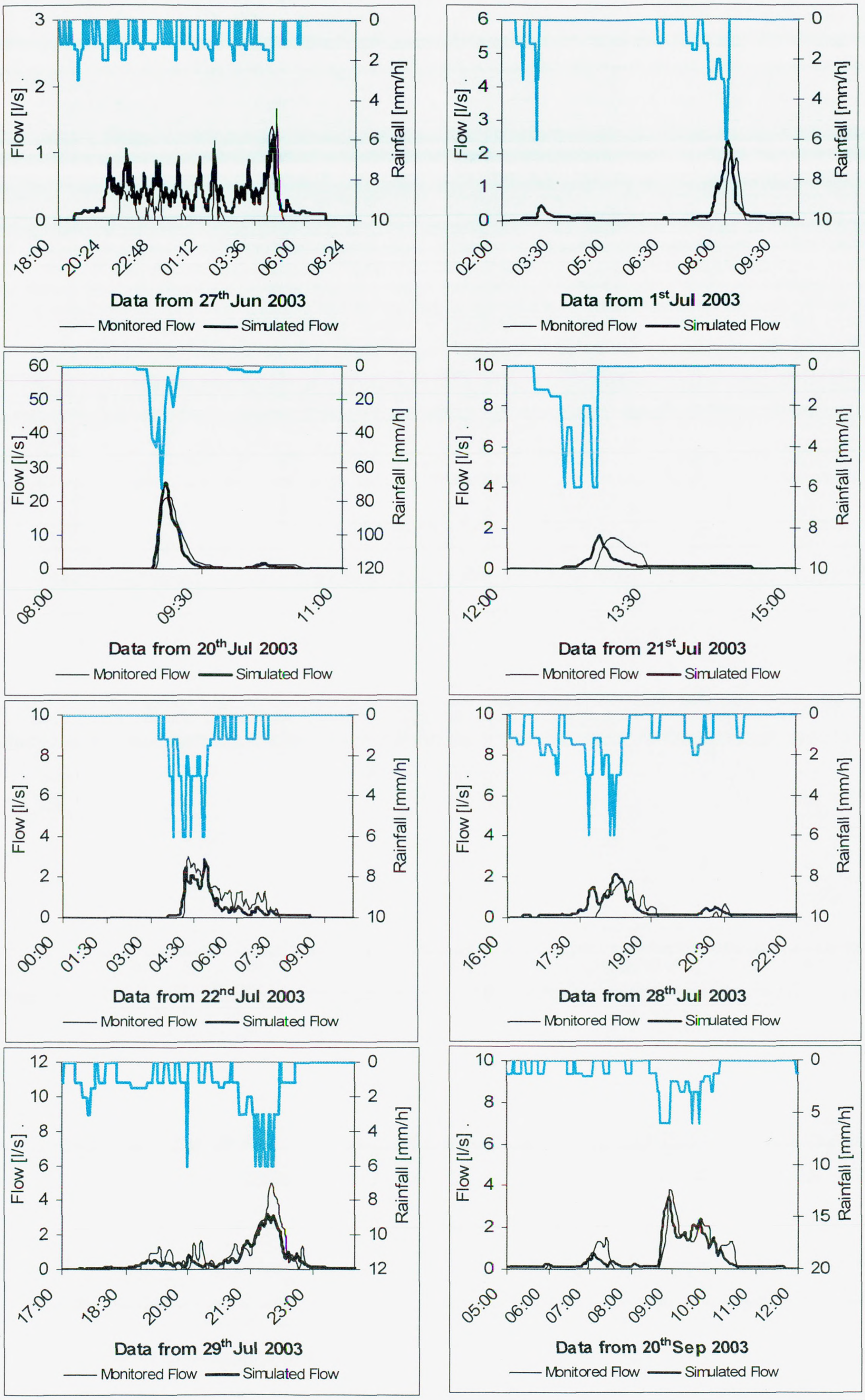



**Figure G-6: Hydrographs of simulated and monitored outflow at Glencarse**

## *Appendix - G.4 Hydraulic Modelling of Broxden*

Modelling the outflow from the infiltration trench at Broxden, was somewhat difficult. The system was found to reduce the flow volume by over 70% and this was combined with long duration of low flows, which was difficult to represent using the features provided in Erwin. However, when discarding the low flow and concentrating on the medium to high storm events a relatively good representation of the flow characteristics was achieved. The best fit was found with F in the order of 35%. When excluding the worst event, F improves to 25%. Figure G-7 shows a comparison of simulated and monitored event volume and Figure G-8 shows a hydrograph comparison of two typical events.

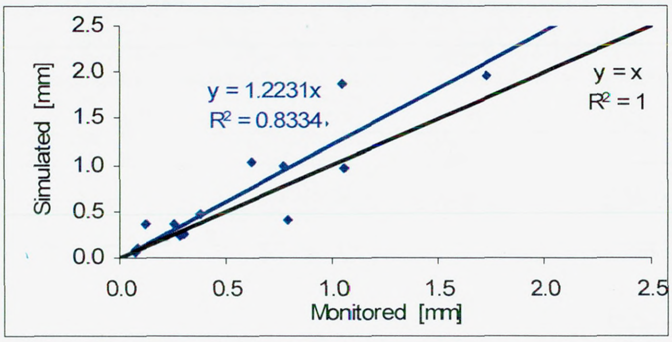



**Figure G-7: Comparison of Event Volume at Broxden, Perth**

The main calibration parameters were the infiltration rate of the base and sides. The porosity was set to 30% and the level discharge relationship was input using monitored data.

| Parameter | Unit | Value |
|---|---|---|
| Initial Loss | [mm] | 0.11 |
| Depression Storage | [mm] | 0.39 |
| Initital Runoff | [%] | 30% |
| Final Runoff | [%] | 85% |
| Porosity of Fill Material | [%] | 30% |
| Infiltration Rate Base | [m/s] | $7.5\ 10^{-5}$ |
| Infiltration Rate Sides | [m/s] | $1\ 10^{-3}$ |

**Table G-4: Main calibration parameters to model Broxden**

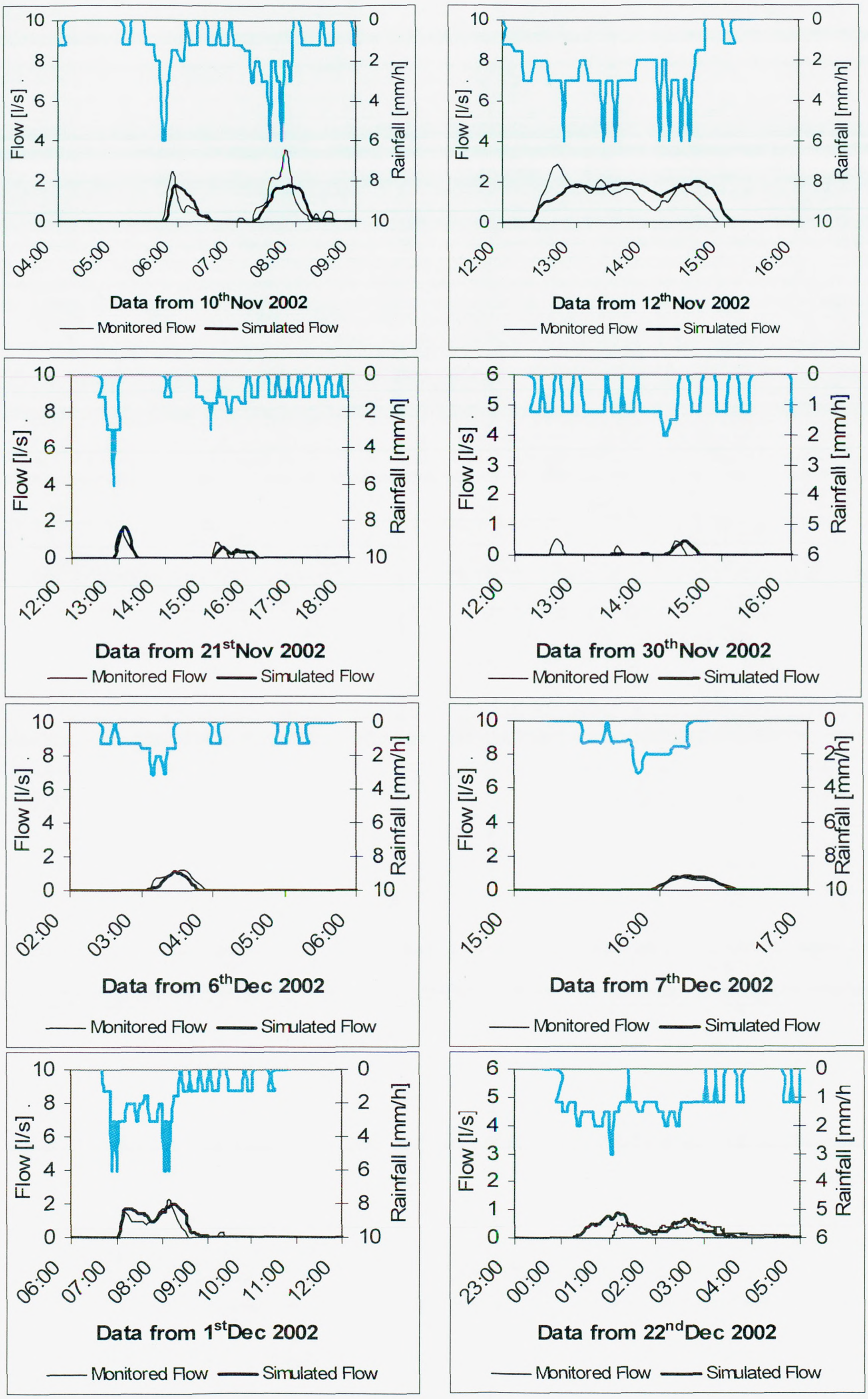



**Figure G-8: Hydrographs of simulated and monitored outflow at Broxden**

## *Appendix - G.5 Hydraulic Modelling of Walker Dam*

A good outflow representation was achieved at Walker Dam and Figure G-9 shows a comparison of simulated and monitored events from the system's outlet. $R^2$ improves from 0.78 to 0.93, when comparing events that were recorded after the site clean-out and this corresponds to an F-value of 22%. Figure G-10 shows a hydrograph comparison of typical events.

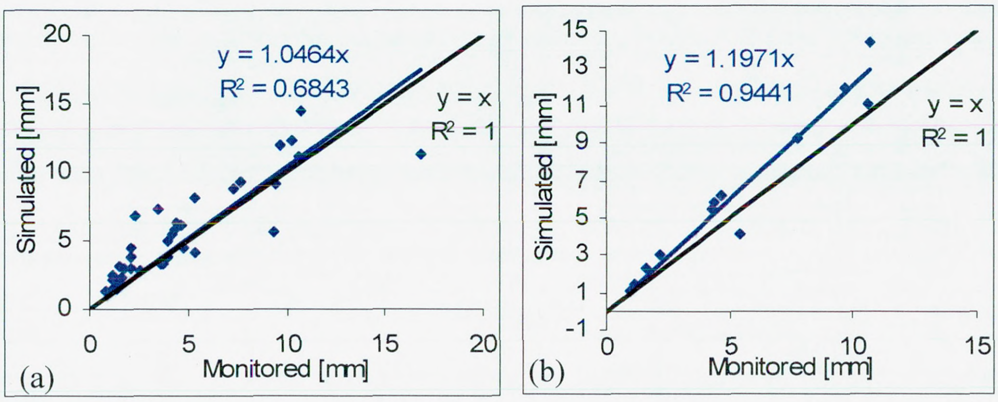



**Figure G-9: Comparison of Event Volume at Walker Dam, Aberdeen; (a) Whole Period, (b) After Cleanout**

The main calibration parameters were the infiltration rate of the base and sides, which were calibrated according to measured level data within the trench. The porosity was set to 30% and the level discharge relationship was input using monitored data.

| Parameter | Unit | Value |
|---|---|---|
| Initial Loss | [mm] | 0.5 |
| Depression Storage | [mm] | 0.6 |
| Initital Runoff | [%] | 25% |
| Final Runoff | [%] | 80% |
| Porosity of Fill Material | [%] | 30% |
| Infiltration Rate Base | [m/s] | $5\ 10^{-8}$ |
| Infiltration Rate Sides | [m/s] | $2.5\ 10^{-6}$ |

**Table G-5: Main calibration parameters to model Walker Dam**

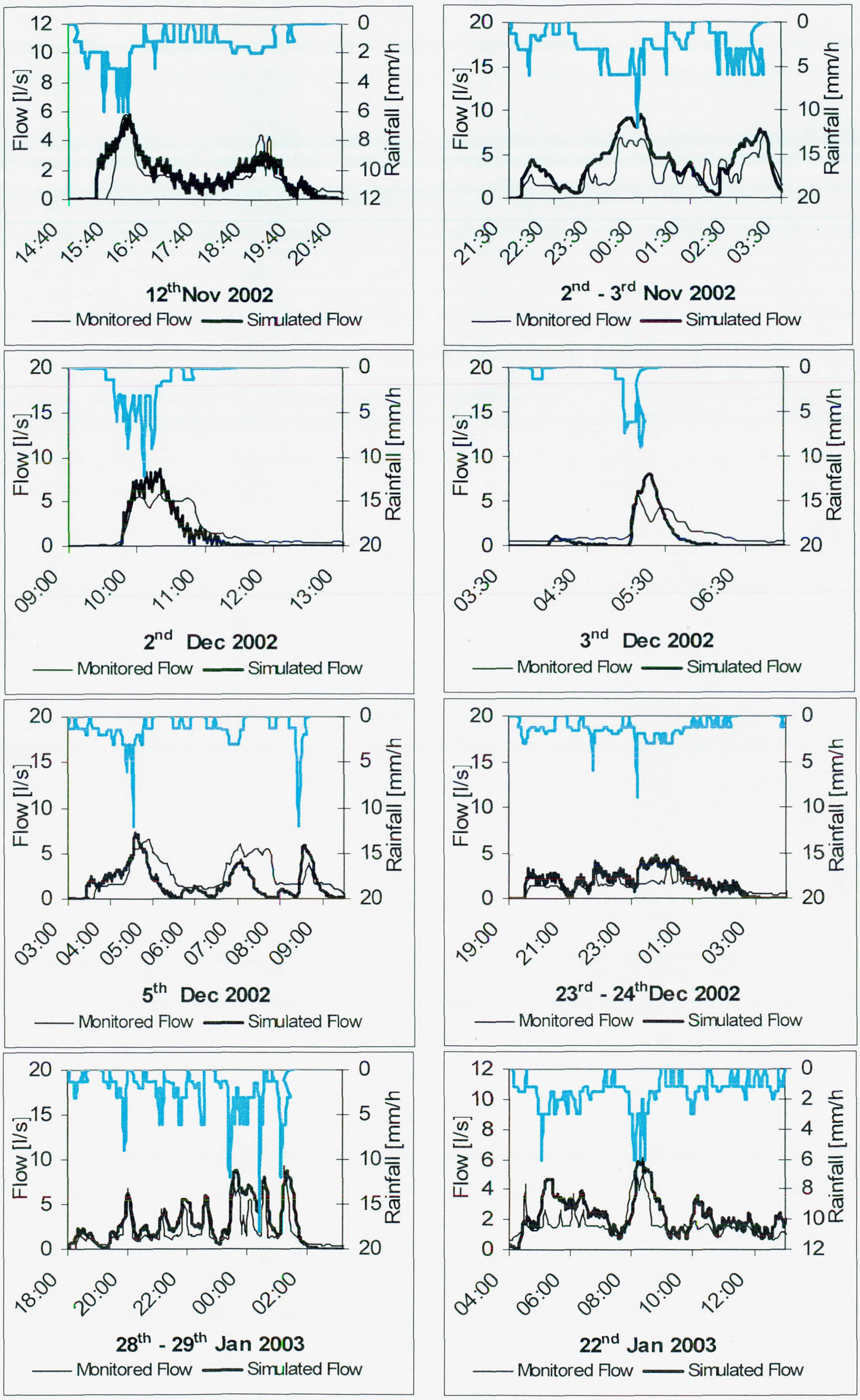



**Figure G-10: Hydrographs of simulated and monitored outflow at Walker Dam**